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SEP 19 1958

# THE STRUCTURAL ENGINEER

THE JOURNAL OF THE
INSTITUTION OF STRUCTURAL ENGINEERS

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# Maxweld reinforces it

The reinforcement fabric used in the roads at the Marchwood Generating Station (C.E.A. Southern Division) was supplied by Richard Hill. Do you need reinforcements? Then call up the Maxweld man! He can give you all the facts on the type and quantity of fabric you'll need plus a rough idea of the cost. And he's backed by the

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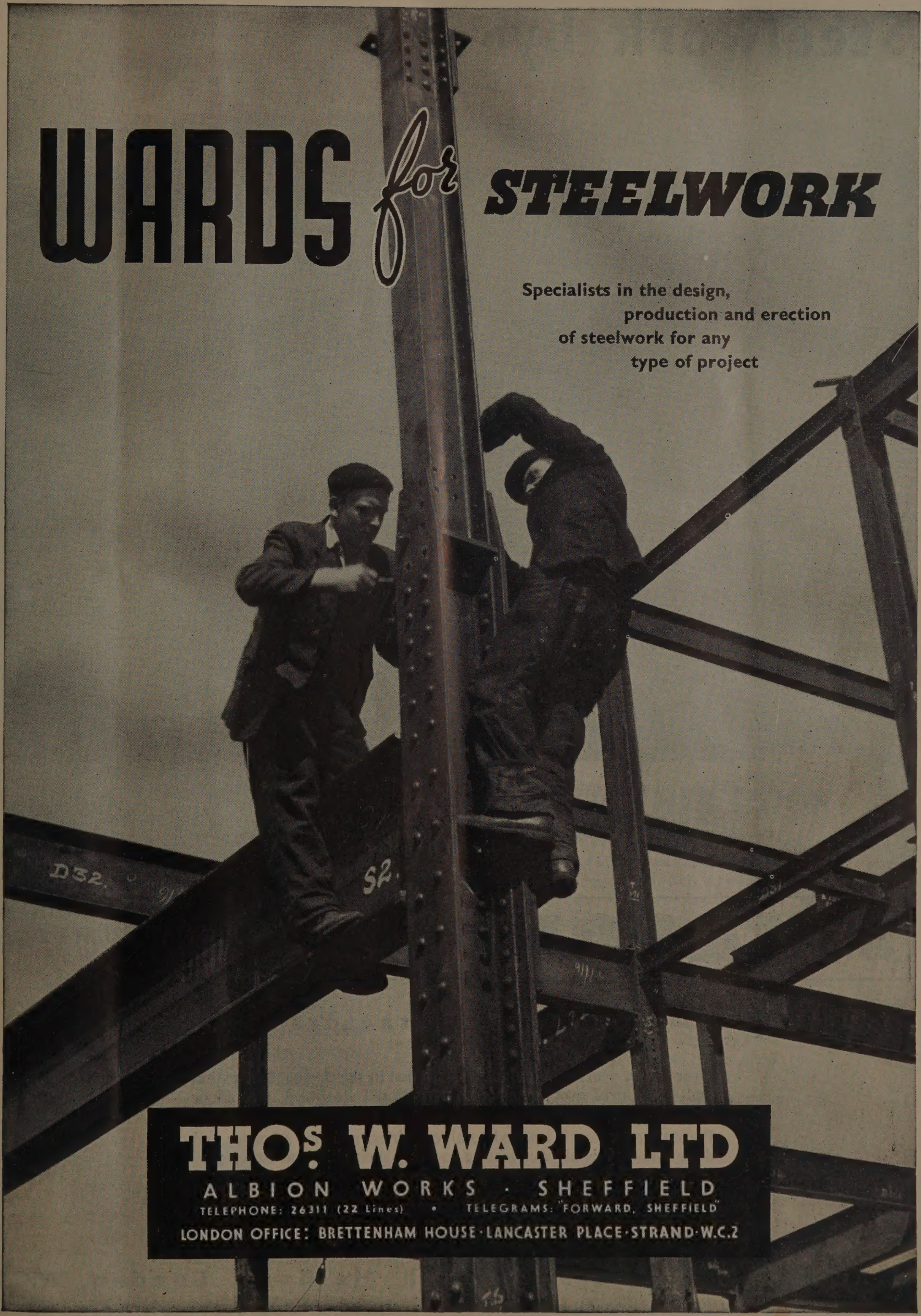
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British Railways, Lincoln. Modernisation of Holmes Yard Goods Depot. Structural Steelwork for Perambulation Shed.

*The works are under the general direction of A. K. Terris, M.I.C.E., Chief Civil Engineer, Eastern Region British Railways, with C. H. Dobbie & Partners as the Consulting Engineers and William Moss & Sons Ltd., as the main Contractors.*

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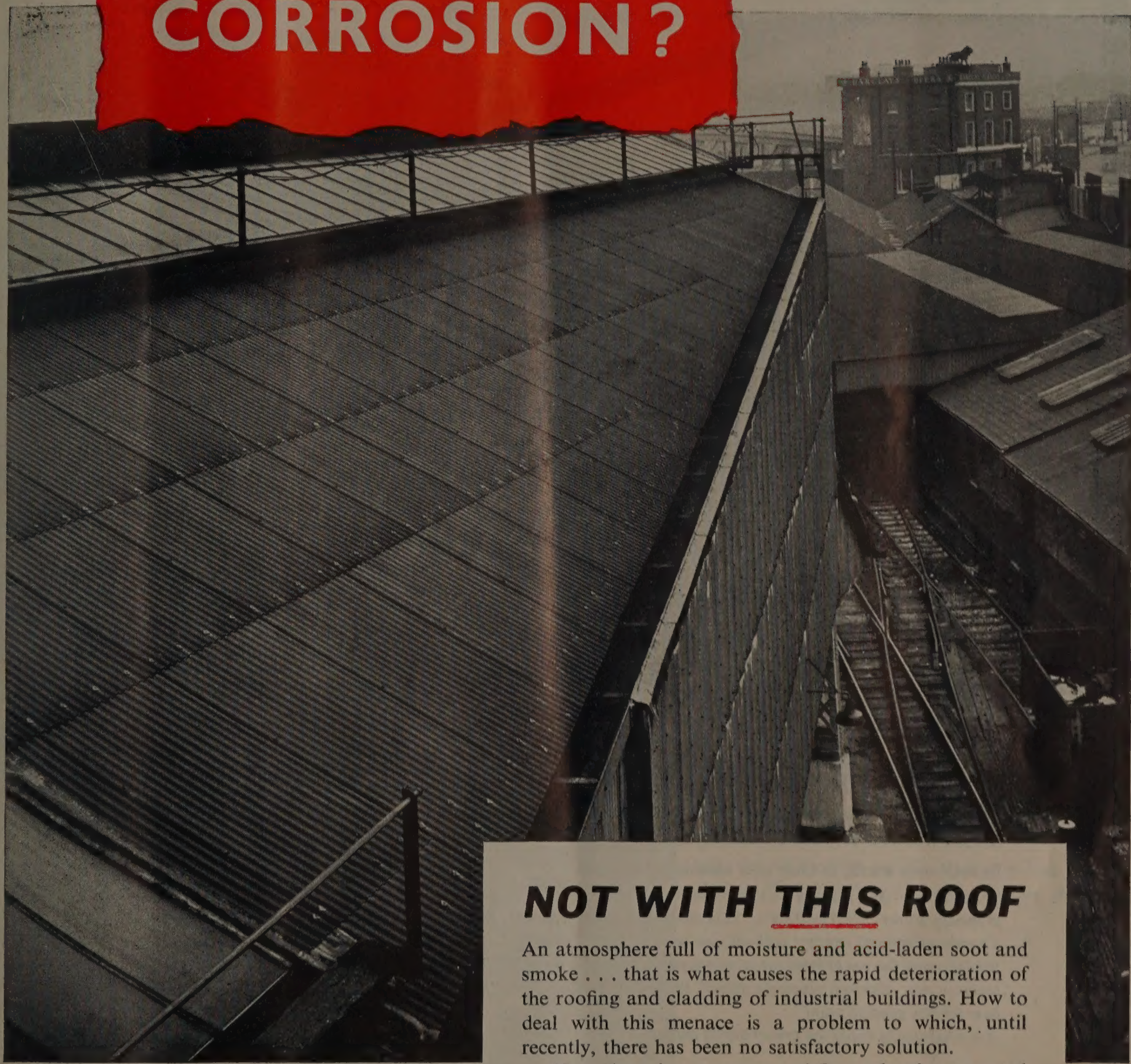
# CORROSION?



## NOT WITH THIS ROOF

An atmosphere full of moisture and acid-laden soot and smoke . . . that is what causes the rapid deterioration of the roofing and cladding of industrial buildings. How to deal with this menace is a problem to which, until recently, there has been no satisfactory solution.

Now there is an answer — ' Corroplast ' corrugated sheets *resist corrosion*. Experience has proved that after many years' exposure in the most severe conditions, ' Corroplast ' sheets do not deteriorate.

These strong, rigid, laminated plastic sheets are economical, too. They last longer than any other sheets, need no maintenance, and are free from breakages in transit.

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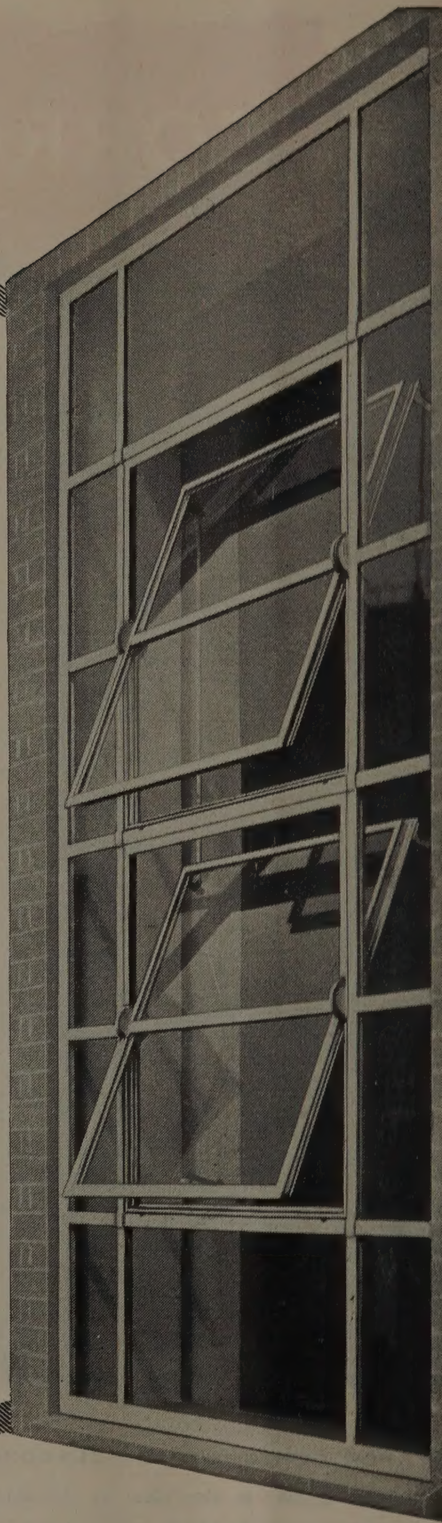
# How HOPE'S made quite sure RUSTODIAN is the primer for galvanized steel windows



Rustodian has convinced Henry Hope & Sons Ltd., that there is nothing like a Calcium Plumbate Primer for galvanized metal window frames. Hope's are one of the largest makers of galvanized metal window frames in this country.

**In their own words, in their own catalogue, they say:—**

*"PAINTING. As a priming coat we recommend the use of calcium plumbate primer 'RUSTODIAN'. This paint will adhere firmly to a newly galvanized window without any prior etching treatment. We deprecate the use of mordant solutions applied to windows on a building site. Calcium plumbate primer 'Rustodian' forms a good base for any good oil paint. It is advisable to apply the primer before glazing."*


The exceptional adhesion and rust inhibiting properties of Rustodian are commending it to many other manufacturers, public authorities and architects for all iron and steel and galvanizing. Rustodian is manufactured under British Patent No. 574826.

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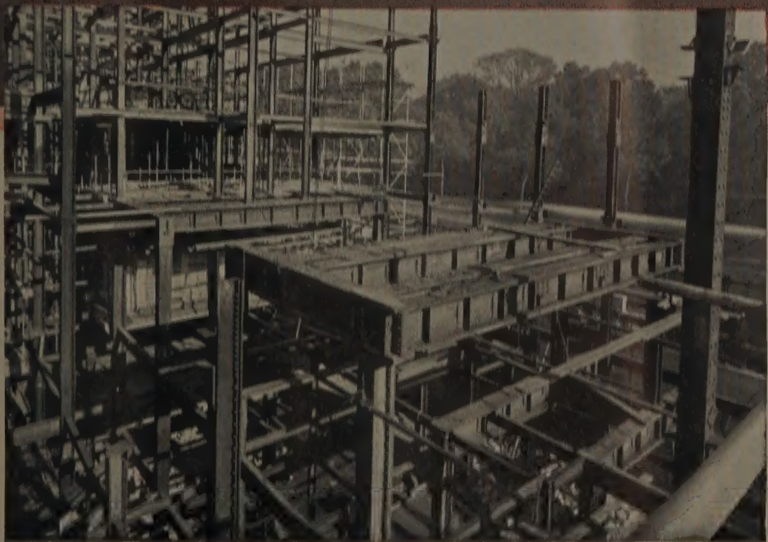


On the right of the top illustration is Iron Trades House, built by Dorman Long before the war.

Architects: Wimperis, Simpson & Fyffe.
Consulting Engineers: R. T. James & Partners.
General Contractors:
Holland & Hannen and Cubitts Ltd.

STRUCTURAL STEELWORK

The illustrations show the office building in Grosvenor Place being erected for Associated Electrical Industries Limited. Steelwork by Dorman Long (Bridge & Engineering) Ltd.

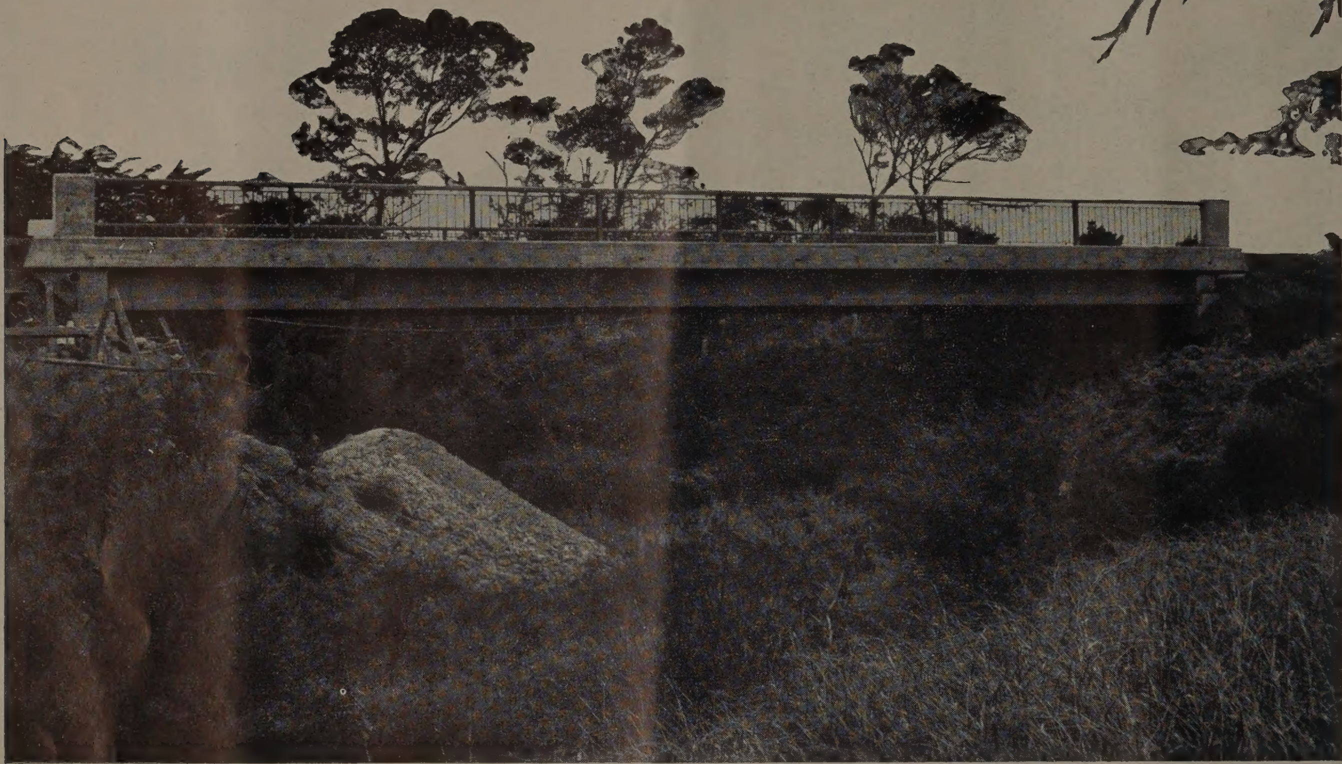
DORMAN LONG

DORMAN LONG (*Bridge & Engineering*) LTD. DORMAN LONG (*Steel*) LTD.

*All steel fabricated by us is produced in our own works and, when required, is erected in any part of the world.*

NO. 13 OF A SERIES SHOWING TECHNICAL DEVELOPMENTS IN CONCRETE CONSTRUCTION

# Bridge Building — the new way



*Prestressed concrete bridge on Kumeu-Helensville main highway, North Auckland, New Zealand*

This is one of over fifty prestressed concrete bridges in New Zealand pioneered by B & B Concrete Co. Ltd. using Johnsons wire. Varying in span from 30 ft. to 120 ft., they have drastically cut the costs of bridge building in terms of time, money and traffic interference. Speed was essential on the Kumeu contract. The first of the beams was delivered to site at the end of October 1955, and the bridge had to be open for the Christmas holidays. For further details write for leaflet.

*Contracting Authority :* Ministry of Works, Auckland.

*Design :* Pre-stressed Concrete (N.Z.) Ltd.

*Sub-Contractors :* B & B Concrete Co. Ltd.

Johnsons were in at the start with wire for prestressed concrete. Their specialist designers studied and worked with the Continental pioneers in the development of the technique, and today Johnsons wire is specified in the vast majority of important prestressing contracts.

**wire was essential —**

**Johnsons**

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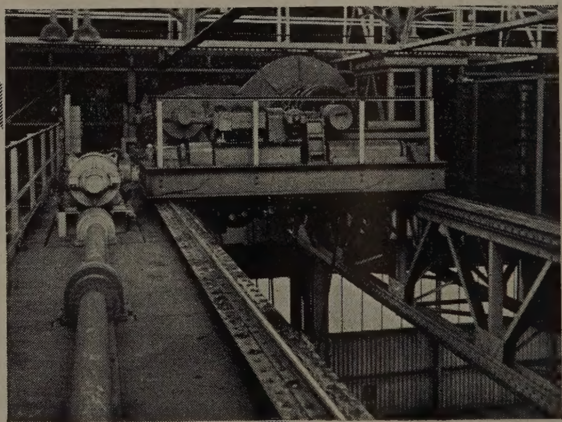
LIMITED-SWING CRANES

by

BUTTERLEY



These photographs show a Butterley steel-works crane installed at the works of Appleby-Frodingham Steel Company for handling plates with a magnet beam.

All gearboxes are of welded construction and the gearing and bearings are designed for long and arduous duty.

Quietness in operation is a feature of Butterley cranes, and limited swing enables the driver to carry out his duties with safety, speed and precision.

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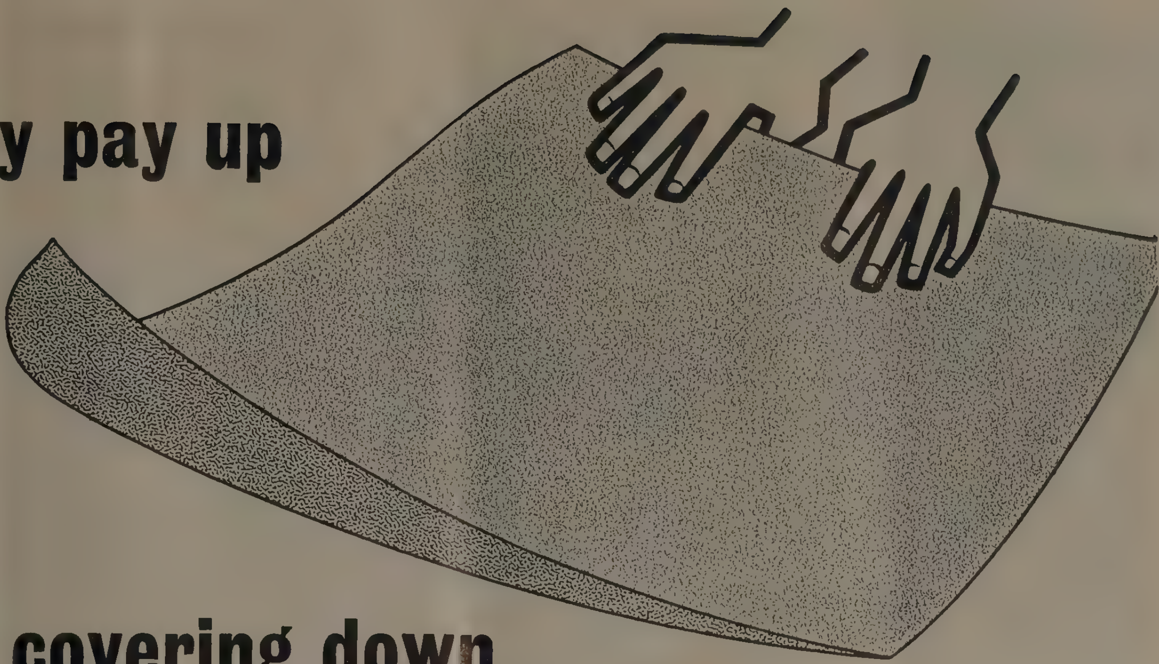
EC 32

# why pay up



# for covering down

***when there's***

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The construction of a surrounding wall to retain the adjoining Queen Victoria Street, Cannon Street and Bucklersbury was greatly facilitated by the use of our bored piles.

The close proximity of important buildings including the Mansion House and services such as the Underground Railway made it necessary to reduce noise, vibration etc., to an absolute minimum.

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**STRUCTURAL STEELWORK BY FINDLAY**

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## *Sellafield*

One of the Pile Buildings with Blower Houses under construction at Windscale Works, Sellafield.
(By kind permission of United Kingdom Atomic Energy Authority)

UNDER DIRECT CONTRACT WITH THE MINISTRY OF WORKS ALEX. FINDLAY & CO. LTD. MOTHERWELL HAVE SUPPLIED, DELIVERED AND ERECTED OVER

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# BOOTH ROAD BRIDGES

## *for the nation's new roads programme*


Two of the main all-welded steel girders for the Cuerdale Road Bridge, Preston-by-Pass, being transported to site.

The work is under the supervision of Mr. James Drake, B.Sc., M.I.C.E., M.I.Mun.E., County Surveyor and Bridgemaster to the Lancashire County Council.


One of two riveted Steel Girders 90ft. 0ins. × 6ft. 4ins. each weighing 20 Tons for the Bardsley Bridge.

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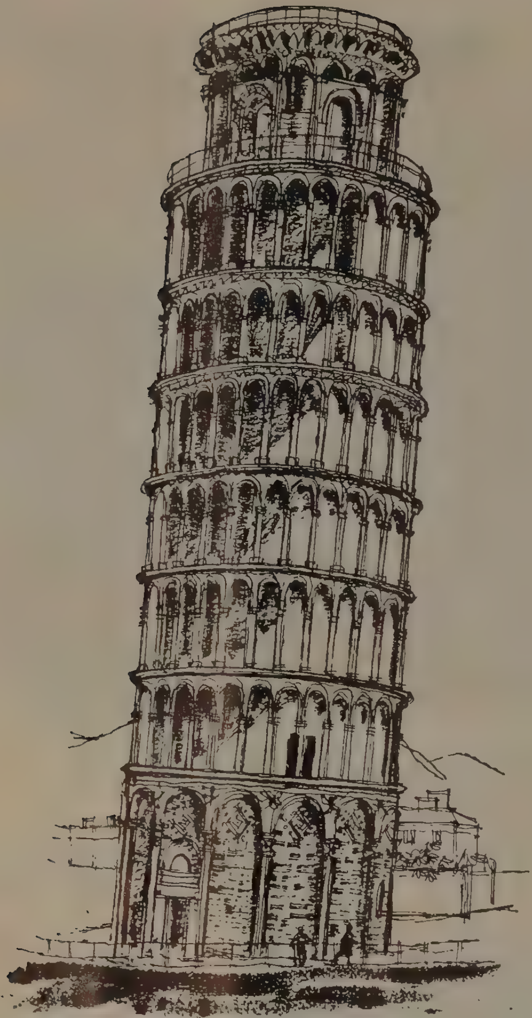

# This couldn't have happened with a DOUGLAS site investigation!

A weak foundation brought world fame to the Campanile Tower of Pisa which now leans 14ft. out of upright.

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*This test accurately measures the shear strength of soils under known stress conditions. On this information can depend the whole foundation design.*

**CONSOLIDATION TEST**

*This apparatus is used to estimate the amount of settlement of a structure on clay soils.*

**INDEX PROPERTIES OF SOILS**

*The illustration shows apparatus used to provide information on the state of materials in the field and to assess their probable behaviour under various conditions. Another vital factor when considering foundation engineering.*

**DOUGLAS** SOIL MECHANICS & FOUNDATION ENGINEERING DEPARTMENT

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*Photo: Steel Company of Wales.*

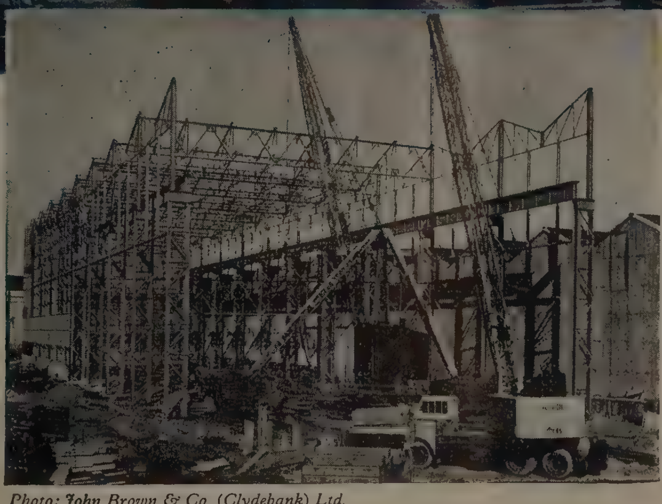

*Photo: John Brown & Co (Clydebank) Ltd.*

# *Steelwork for Industrial Buildings*

Steel and shipbuilding are two of the many industries served by ARROL. These photographs show examples of a common trust in ARROL'S ability to meet the exact requirements of any modern industrial project.

*All types of Steel Framed Buildings; Fixed and Opening Bridges; Cranes and Mechanical Engineering work; Dock Gates; Sliding and Floating Caissons; Compressed Air Locks; Hydraulic Machinery; Pipe Lines, Surge Tanks, Sluices and other equipment for Hydro-Electric Projects.*

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*Model of the Atomium*

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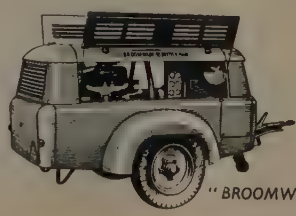
This Atomium, symbolising the constructive rather than the destructive forces at work in the world today, was conceived and built in a spirit of international co-operation. "BROOMWADE" Air Compressors helped by supplying compressed air for drilling and riveting the steel framework and alloy sheeting.

A cut-out model of the latest "BROOMWADE" WR 120 Rotary Portable Compressor is on view at the Group Stand, British Industries Pavilion, Block C.



"BROOMWADE" WR 120

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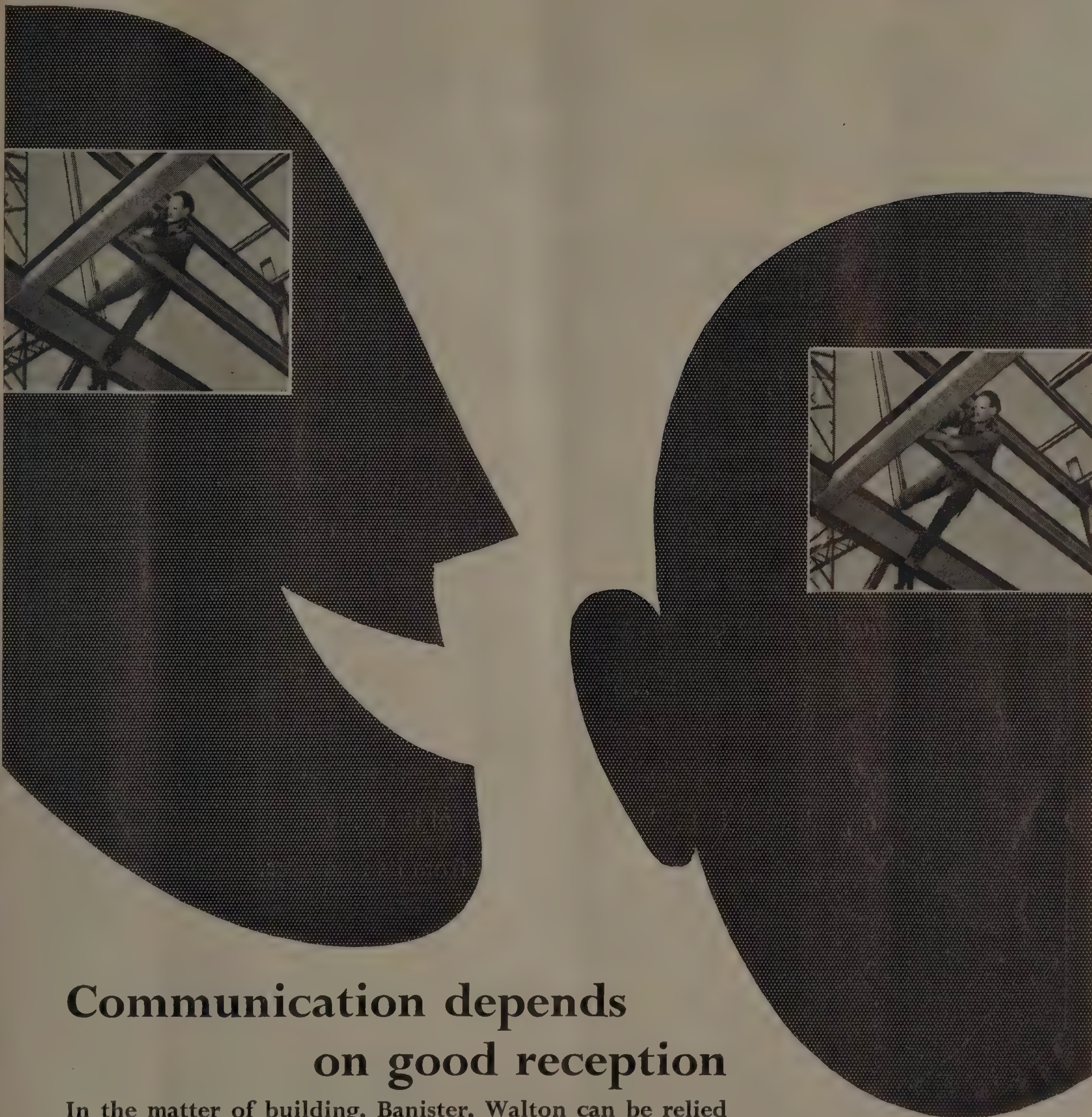


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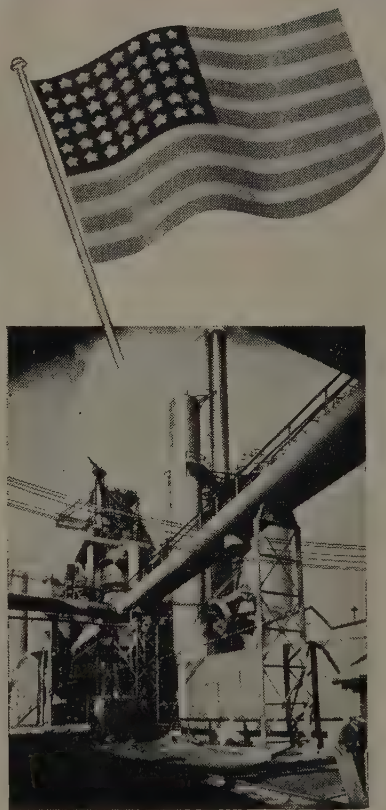

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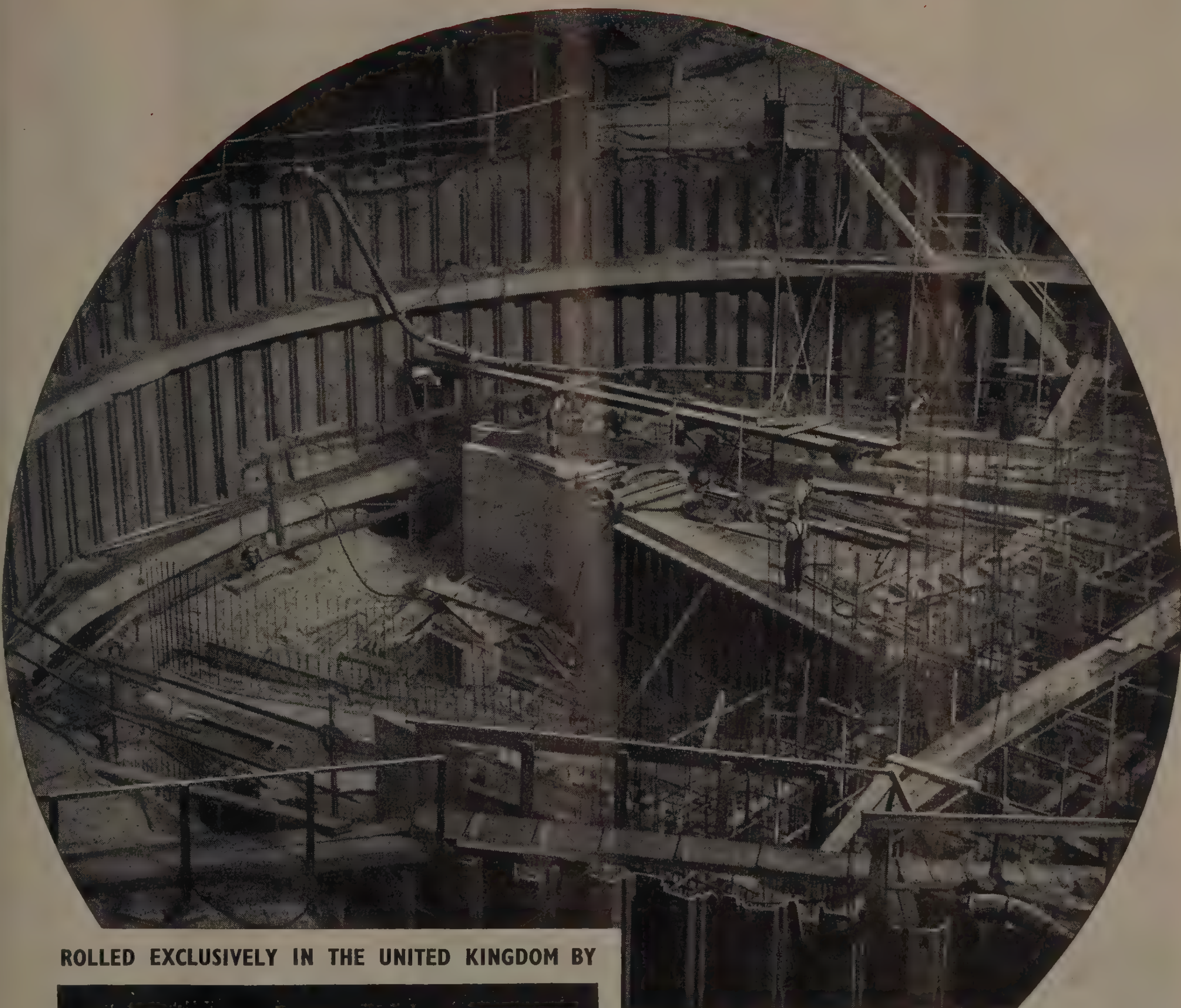

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This photograph illustrates Pymmes Brook culvert, Tottenham.

Other work in progress includes tunnelling, sewerage, concrete structures, factories and domestic buildings.

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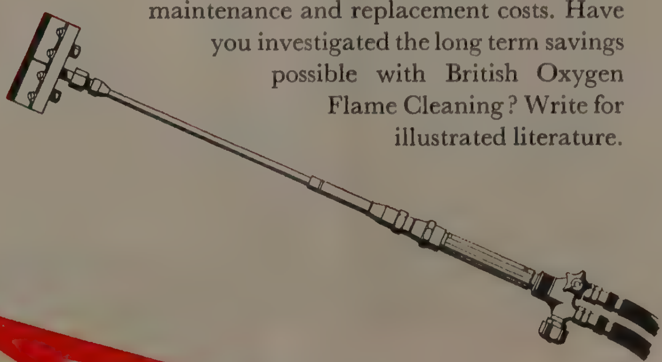
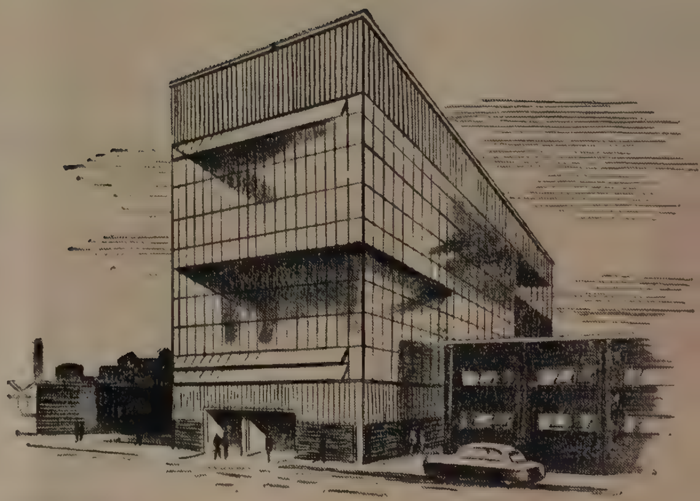


BOC BRITISH OXYGEN

British Oxygen Gases Ltd., Industrial Division, Spencer House, 27 St. James's Place, London, S.W.1

# Landmarks of Today



The new Boiler House for the I.C.I. Kynoch Works at Witton, Birmingham, is an impressive construction for which we fabricated and erected the steelwork. Its striking proportions provide yet another of to-day's landmarks.

*Illustration by courtesy of Imperial Chemical Industries Limited, Metals Division.*

*Steelwork to the design of Babcock & Wilcox Limited, London and Renfrew.*

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PLATES AND SECTIONS

*The framework of the building (left) was constructed to a large extent from Appleby-Frodingham sections and plates, (right) M.S. "Bergensfjord" built or Den Norske Amerikalinje A/S of Oslo has a gross weight of 18,750 tons. Considerable quantities of Appleby-Frodingham plates and sections were supplied to Swan, Hunter, & Wigham Richardson, Limited, the builders of this ship.*

THE UNITED STEEL COMPANIES LTD

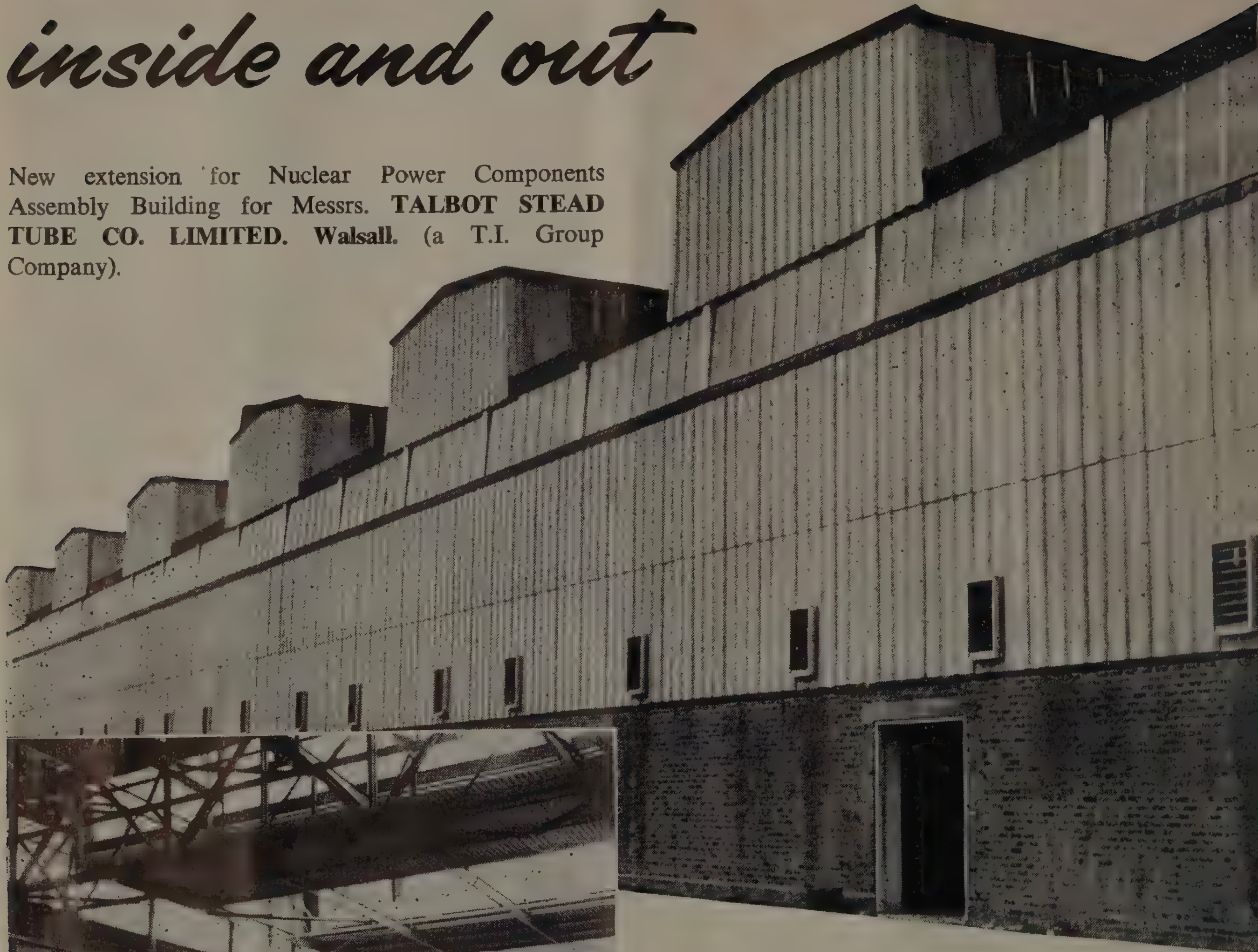
APPLEBY FRODINGHAM STEEL COMPANY

*A Branch of The United Steel Companies Limited* SCUNTHORPE LINCOLNSHIRE

AF 164

# CLEAN LINES
## *inside and out*

New extension for Nuclear Power Components Assembly Building for Messrs. **TALBOT STEAD TUBE CO. LIMITED. Walsall.** (a T.I. Group Company).



Exceptionally clean lines are the feature of this building, one of the many modern factory extensions for which the Rubery Owen Structural Division has been responsible.

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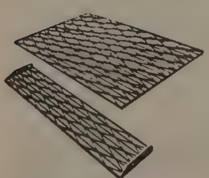

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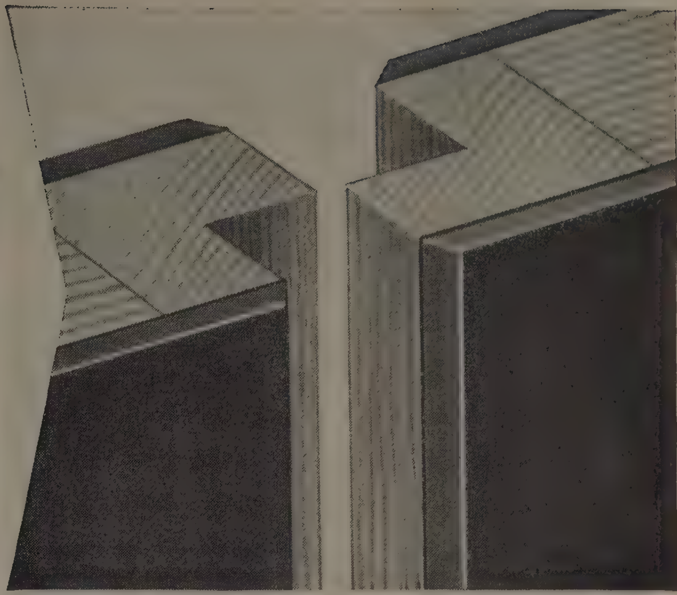

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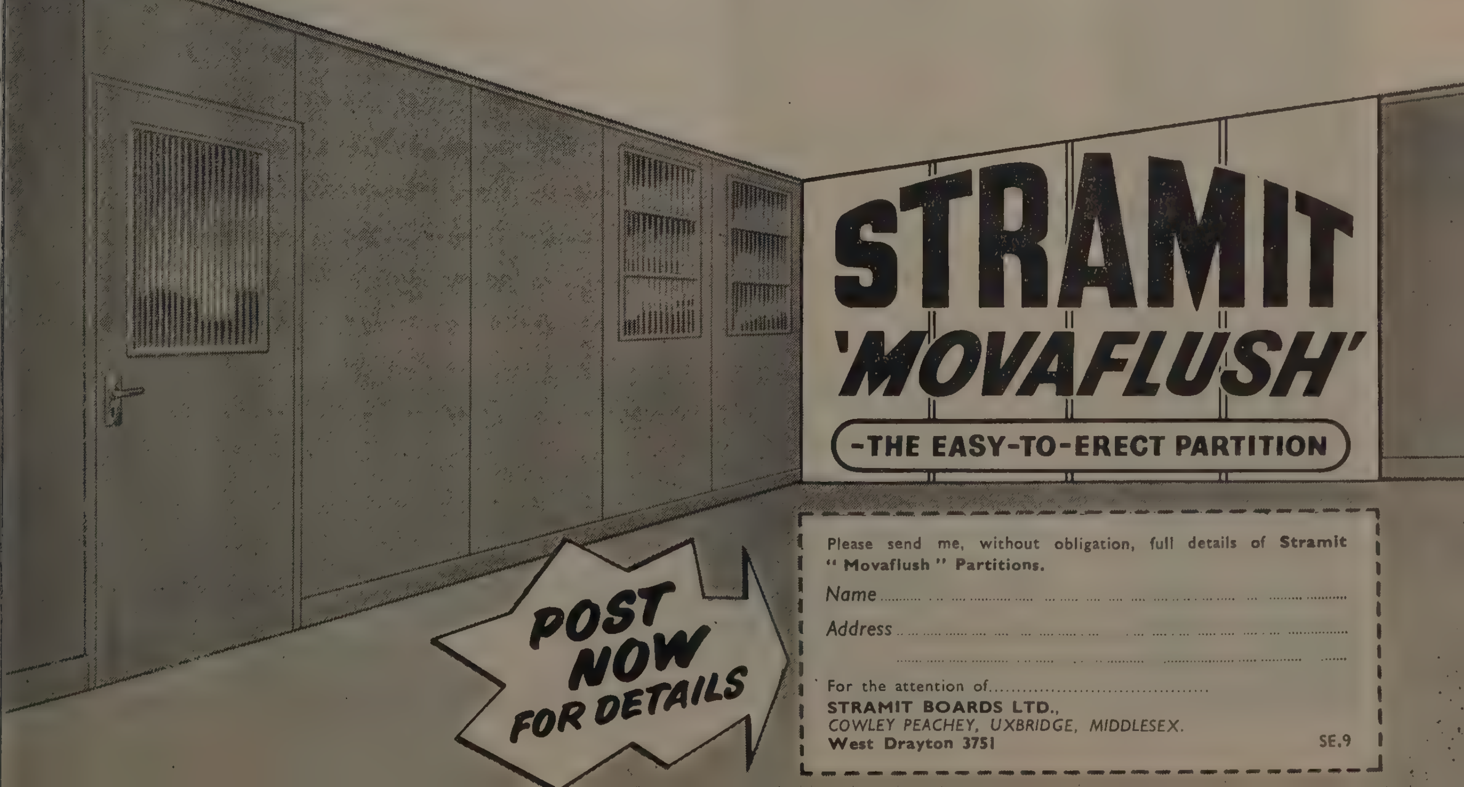


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# THE STRUCTURAL ENGINEER

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# Progress on Britain's First Motorway


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# Maintenance of Plymouth Breakwater*

By T. W. Riley, A.M.I.Struct.E., A M.I.C.E.

## Introduction

The Breakwater in Plymouth Sound was designed by Mr. John Rennie in 1806, and he was responsible for its construction until his death in 1821, when his son, George, assumed the responsibility.

The Breakwater which is isolated, is approximately two miles from the City of Plymouth, 1,600 yards from the Cornish shore to the western end, and 1,000 yards from the eastern end to the Devon shore, and was constructed of random blocks of limestone which were dropped promiscuously from sailing vessels. The work on the Breakwater commenced in 1811. During its construction its height was raised from 10 feet above L.W.E.S.T. to 2 feet above H.W.E.S.T. and the possibility was foreseen by Mr. John Rennie of paving the top, sea and landward slopes of large block of dressed limestone, and this was carried out after a great storm that occurred in 1824. (Fig. 2).

It was found that the limestone facing was insufficient to withstand the force of the sea, and additional granite bands were incorporated in the top and seaward slopes. The toe of the seaward slope constructed at low water was frequently disturbed, particularly at the western end, where it received the full force of the westerly gales. This was eventually strengthened by the construction of a granite buttress, the blocks of which were dressed, dovetailed and lewis bolted. This buttress commenced at the western end and was continued to a lesser degree throughout the length of the seaward side to the eastern end. To protect the lighthouse at the western end, an angular buttress was built in 1838, twenty-seven years after the commencement of the work. Severe damage in 1848 was responsible for the construction of the angular buttress at the eastern end, and it was not until 1885 that the reconstruction work on the buttresses was finally complete for the whole of the Breakwater. It might appear that the buttresses had taken an immense amount of time to construct, but when it is realised that the foundations for the buttresses were at or below the level of low water, and the time for working and preparing the foundation was so limited, and that often the sea destroyed all the preparations that were necessary for setting out the blocks, the time taken will be viewed with tolerance, with the knowledge that 265 feet of the western arm buttress was destroyed in a gale in 1867.

The limestone for the work was obtained from quarries at Oreston, Devon, approximately three miles from the site, and the granite from Par, in Cornwall.

**Fig. 1.—Aerial View of Plymouth Breakwater, taken from the west at low water.**

## Maintenance

After the completion of the buttresses and the facing to the Breakwater, it was considered necessary to protect the toe and the seaward slope, and it was decided in 1871 to construct concrete blocks to form wavebreakers, and two were made ten feet in diameter and 5 ft. 4 ins. deep, weighing 28 tons each. (Fig. 3). These were placed on the foreshore (Fig. 2) along the south of the western arm in the summer of that year, but during the following winter they were washed over to the north slope. The impression formed from this was that the cylindrical shape was most unsuitable for use where there is an inclined slope where the block could be, and quite possibly was, turned over and rolled up the slope and over the Breakwater fairly easily, in contrast to the fact that a large limestone block of approximately 35 tons weight which was deposited at the same time was left undisturbed.

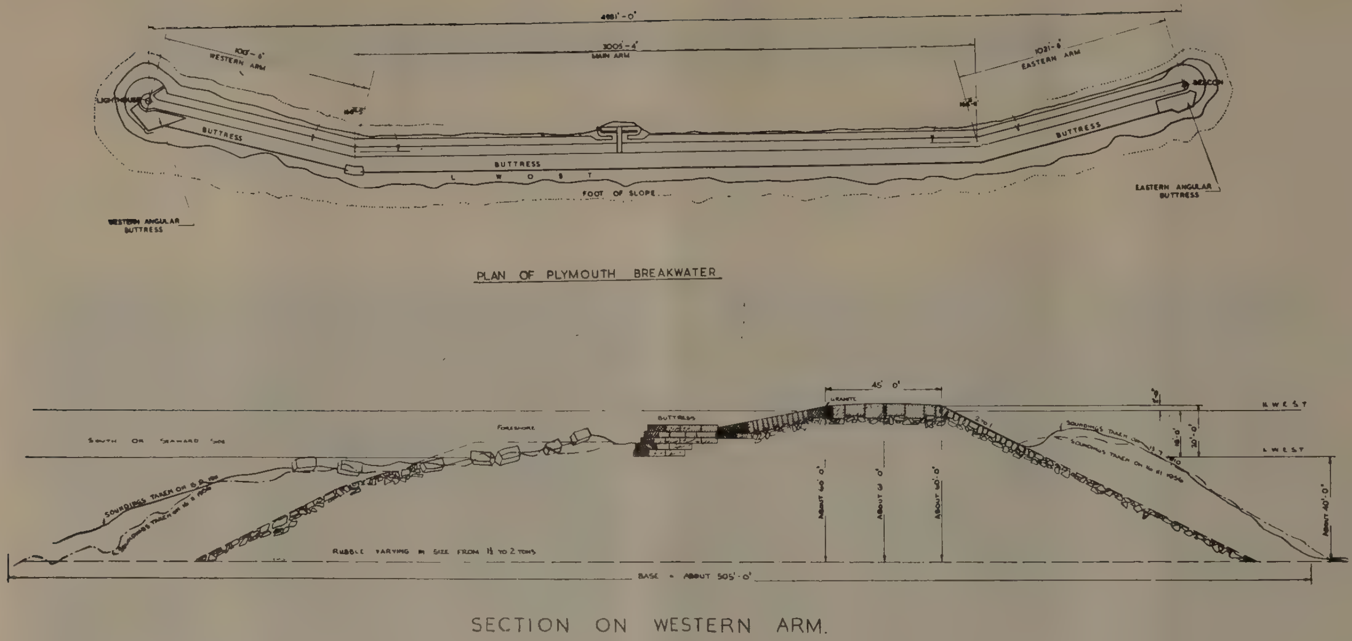


**Fig. 2.—Plan of Plymouth Breakwater and section on western arm.**

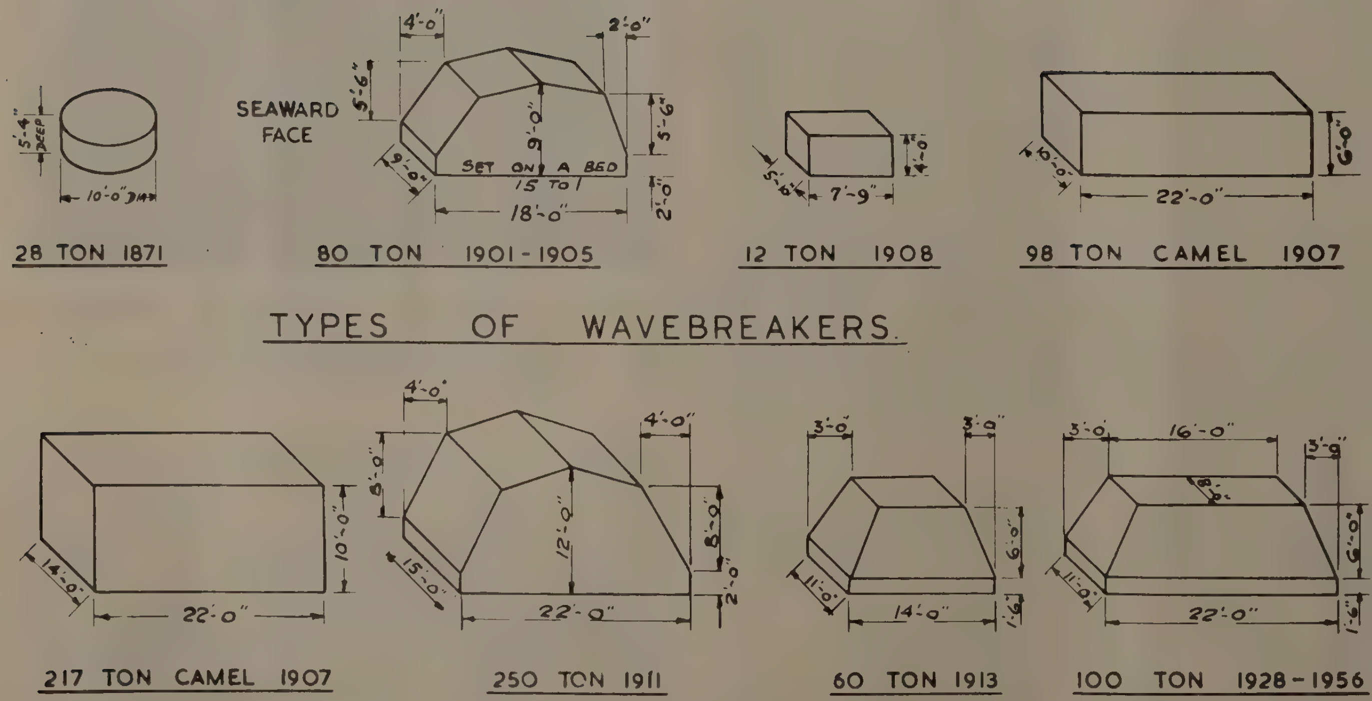


**Fig. 3.—Types of Wavebreakers.**

Fig. 4.—View from Lighthouse looking south-east, showing Buttress along south side of the western arm, with Wavebreakers on foreshore. Taken at low water.

Fig. 5.—View taken from Lighthouse looking north-east, showing the amount of stone washed over.

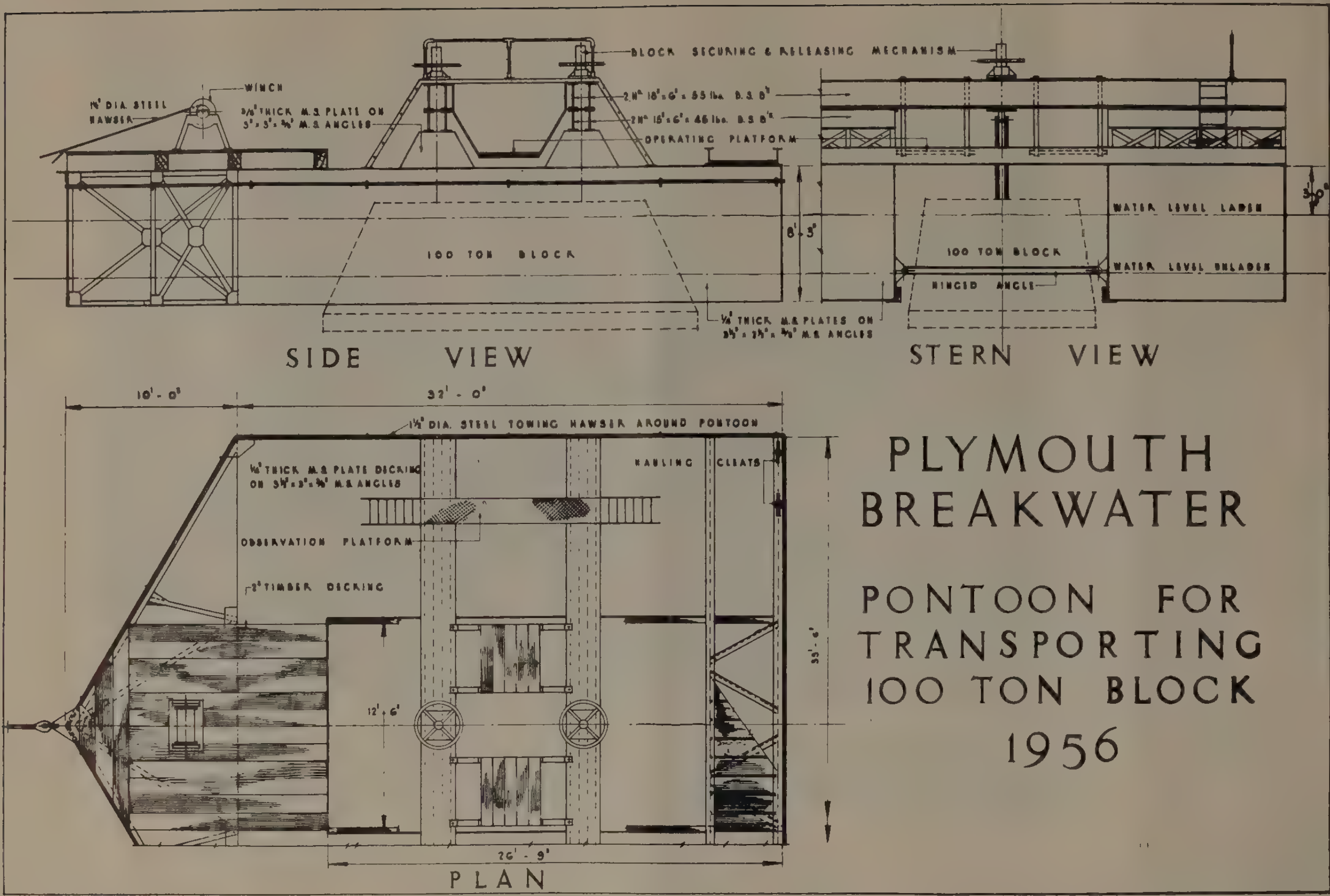


**Fig 6.**

Whether the loss of the two concrete blocks discouraged the engineers at the time is unknown, but twenty nine years were to elapse before the second experiment of making concrete wavebreakers was attempted. In the meantime approximately 3,000 tons of limestone was dropped each year to replace the material that was being displaced from the south to the north side by the gales.

It was to prevent this loss that 8 No. 80 ton wavebreakers were constructed, 18 feet long by 9 feet wide, and 9 feet high, trapezoidal in section, and were cast on the foreshore south west of the lighthouse, but not before some difficulties had to be surmounted. In the first attempt, shuttering of 12 ins. x 6 ins. timbers were prepared and framed together, transported to the site and secured in position on the foreshore with chains to suitable anchorages at low water. This was found to be impractical as the formwork rose with the tide. A further attempt was made with additional means of anchoring, and this again failed. Finally, shuttering constructed of boarding, fastened to steel angles and suitably braced was found to answer the purpose. A total of forty eight 80 ton wavebreakers were constructed between the years 1901 and 1905, and all were placed to protect the western arm. A gale in 1906 carried two of these blocks across from the south of the lighthouse to the north side, and the remainder were all moved bringing them closer together, leaving a wide breach unprotected south of the angular buttress. To fill this breach, an opportunity arose of using steel tanks, constructed of $\frac{3}{16}$ths to $\frac{1}{4}$ in. mild steel plate, well braced internally with angles, and were of the following dimensions: one, 22 feet long by 14 feet wide, and 10 feet high, weighing 12 tons: twenty seven tanks, 22 feet long by 10 feet wide and 6 feet high, weighing six tons each. The largest tank when filled with concrete would weigh 217 tons, and the smaller tanks 98 tons.

It was considered that these "Camels" as they were called had every advantage, inasmuch they supplied additional weight and stability, could be easily towed out and placed into position farther south of the foreshore, and that the bracing would provide reinforcement. Should the blocks be moved eventually, they would not be responsible for the same loss that the 80 ton blocks had made, for the following reason. When the 80 ton blocks were cast on the foreshore, it was considered that these blocks being embedded on the large boulders that formed the foreshore would both strengthen the block and add to its weight, but, unfortunately, this was not found to be so, when the wavebreaker was removed or overturned, the irregular bottom of the block was found to be more liable to disintegration than the smooth surface of the top and sides.

The 217 ton "Camel" and six of the 98 ton "Camels" were concreted in 1907, and were secured together by lengths of $2\frac{1}{4}$ ins. chain link cable fastened to heavy shackles cast in situ with the block.

In 1908, a further six 98 ton "Camels" were concreted, and 195 12 ton blocks, the latter were 7 feet 9 ins. long by 5 feet 10 ins. wide and 4 feet 0 ins. high. The latter were cast and placed throughout the length of the southern foreshore, and chained together with $1\frac{1}{4}$ in. stud chain cable. In the following year an additional 300 12 ton blocks were deposited.

It was found in this following year, 1909, that the chain link cable did not improve the efficiency of the wavebreakers, when it was discovered after the gales

Fig. 7.—View showing pontoon prior to the release of the block under ideal conditions. Grouted joints of the limestone shown in foreground.

Fig. 8.—View of pontoon after release of block.

Fig. 9.—View of foreshore, showing overturned 100 ton block.

Fig. 10.—View of 100 ton blocks on casting bed of River Plym, Oreston, Devon.

of the 15th and 16th December, that nine of the 98 ton blocks were washed over the western end, and that the 217 ton block had been reduced to 50 per cent of its original weight. The internal bracing of the "Camel" far from providing reinforcement and strengthening the block, produced the opposite effect. The metal had corroded, and the pounding of the sea caused disturbance both of the metal and the concrete, gradually disintegrating the latter.

The displacement of the 98 ton blocks from the western end, drew attention to the problem that at this end larger blocks were necessary to resist the force of the sea, and provision must be made for the wearing of the block.

It was proposed in 1911 to construct three 250 ton blocks, to be cast in situ, the concrete being mixed manually, and each block was to be completed during one tide. The first was constructed on 28th July, but owing to unfavourable weather conditions, it was only possible to complete 210 tons. Measures were taken to add another 40 tons of concrete by inserting steel lewis bolts to bond and tie the new concrete to the old, but a swiftly rising sea swept the concrete and the shuttering away.

The second block cast on 11th September met with greater success. The concrete was mixed by hand by four gangs of workmen, making a total of 72 men.

In 1912, of a total tonnage of wavebreakers constructed, 4,810 tons were still in position on 8th December, this figure representing a percentage of 74 per cent of the original deposited. Of the 12 ton blocks, only 3,859 tons were still in position of a total of 11,340 tons, representing a loss of 66 per cent. The figure for the larger wavebreakers confirmed the impression gained at that time, that the wavebreakers were efficient, and that the use of them for the maintenance and protection of the work should be continued. It was also suggested at the time, that the wavebreakers should vary in size relative to their position on the Breakwater, at the extreme western end where the force of the seas were strongest, they were to be of 250 tons, and for the remainder they were to be of 60 tons.

Three 250 ton blocks were cast in situ in 1912, and these were the last to be constructed on the site.

In the next year, it was decided to use two "Camels" and with them to construct a pontoon to transport the 60 ton blocks from the casting beds on the bed of the River Plym at Oreston. The casting and placing of these blocks were continued, except for the period of the First World War, and from 1925 to 1929, until 1934. In the meantime the pontoon which had been in service from 1913 had reached a condition when it was considered desirable to obtain another to replace it. Designs were prepared, and estimates made for the new pontoon which consisted of two steel tanks, divided into compartments, sheeted with $\frac{1}{4}$ in. mild steel plates, and stiffened with $3\frac{1}{2}$ ins. x $3\frac{1}{2}$ ins. x $\frac{3}{8}$ in. angles. The tanks were 32 feet long, 10 feet wide and 8 feet 3 ins. deep, and Messrs. Phillips, Shipbuilders of Dartmouth, constructed the craft for the sum of £1,400 and it commenced service in 1928.

Prior to the pontoon commencing its service, the question of the size of the blocks to be constructed again came under review, and it was decided to construct 100 ton blocks, and the first of these were deposited in December, 1928. In 1929, four were deposited south west of the Lighthouse, of which one was destroyed during a gale in December, (when 3,000 tons of material was washed over), and which caused the breaking up of one of the 250 ton blocks.

The casting and depositing of the 100 ton blocks continued on a fairly small scale until 1939 when production ceased, and it was not until 1954, when the question of block laying was reviewed. It was decided that the placing of the blocks should be increased to preserve the pitching of the foreshore, which, if allowed to deteriorate closer than 20 feet from the granite buttress might cause serious damage.

The placing of the blocks were to be in the following sequence, (a) to be placed in the low areas at the western end, (b) to be placed in a line at the western end 20 feet away from the granite buttress, and at 20 feet centres at right angles to the buttresses, (c) placed in line further out and in low areas along the eastern arm of the Breakwater, (d) to continue the 20 foot line of blocks from the western arm to the eastern end of the Breakwater. Finally it was suggested to lay, in addition to the above, a line of blocks end to end, hard against the granite buttress.

In support of this programme 47 were placed in 1954.

In the November following, a gale washed some 3,000 to 6,000 tons of material over the Breakwater, and over 50 per cent of the blocks placed that year had been moved, some to a maximum of 15 feet, one being overturned. Depositing continued the following year when 57 were placed, and is still continuing.

The mix for the concrete is 1 : 3 : 6.

A new pontoon designed and constructed on similar lines to the existing one, was brought into service in 1956, and to prevent corrosion this is cathodically protected. When the blocks are ready to be lifted, the pontoon is towed by a launch and placed over the block, and the block is hooked to the pontoon. As the tide rises the block is lifted from the bed, and a tug then tows the pontoon to the vicinity of the pre-selected site on the Breakwater. Hawsers are then taken from the pontoon by the crew of a dinghy and fastened on the Breakwater. The pontoon is then manually operated by its crew hauling on the hawsers until they reach the mark to which they are to release the block.

The number of blocks that can be placed during a year is limited by the tides and by the weather conditions. They can only be delivered at high tide, and during the hours of daylight.

Grouting of the joints of the blockwork has been carried out for a number of years since 1920, but it was not until 1934 that it was executed in any magnitude. The mixes varying with $\frac{3}{8}$ in. to 2 in. aggregate, making a total of 3,915 tons. What quantity of grouting will be required in the future is problematical, as the joints vary in width and depth, an example of the latter was shown when a bar was inserted a depth of eight feet. The joints between the limestone have probably been increased during their existence by the added effects of the flow of sea water, with the additional abrasive effect of small particles of stone driven upwards by compressed air.

A survey made in 1955 showed that no settlement of any importance, if at all, has occurred on the top, but on the southern slope there are a number of blocks that appear to have subsided for a varying depth up to nine inches, but are immovable.

A recent survey of the number of wavebreakers remaining on the foreshore south of the breakwater

suggests that the placing of the 100 ton blocks since 1928 appears to be the answer to the question of the size of the block to retain the rubble foreshore and to provide protection to the buttresses.

Total Tonnage Deposited :—57,930 tons.
Total Tonnage remaining :—34,443 tons.
Total Percentage Loss :—40.5 per cent.

Earlier mention has been made of the placing of 12 ton blocks, this included the weight of the vehicle carrying the block, and has been deducted in the table below.

In the past, movement of the blocks has been assisted by the lack of preparation of the site to receive a block. Time and tides have prevented the removal of the remnants of blocks previously placed, and blocks were deposited which left them pivoted on the old ones, leaving the underside of the block to receive the full effect of the pressure of the sea. With the aid of mechanical equipment these are now drilled and demolished with the aid of explosives, and thereby providing a reasonably level bed to receive a block.

The present condition of the Breakwater, and of Plymouth Sound fulfils all the expectations of its eminent designer and of his son, and of the Admiralty Engineers who followed them, and by whom it was and is maintained at such little cost.

**WAVE BREAKERS**
**Tons**

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Weight | 250 | 217 | 100 | 98 | 80 | 60 | 40 | 30 | 11¼ |
| Number Deposited | 5 | 1 | 261 | 20 | 48 | 164 | 2 | 4 | 1291 |
| Number Existing | Remains of 1 | 0 | 234 | 5 | 12 | 139 | 0 | 0 | 98 |
| Tonnage Placed | 1250 | 217 | 26100 | 1960 | 3840 | 9840 | 80 | 120 | 14524 |
| Tonnage Remaining | 150 | 0 | 23400 | 490 | 960 | 8340 | 0 | 0 | 1103 |
| Percentage Loss | 88% | 100% | 10.35% | 75% | 75% | 15.3% | 100% | 100% | 93% |

From the above table it will also be observed that a number of the 100 ton blocks which were not destroyed, are missing from the foreshore, having been discharged into deeper waters south of the foreshore. Before blocks are transported to the site, weather reports are obtained as to the suitability of the winds and weather conditions, and on favourable reports being received, operations are commenced. During the time taken to travel from Oreston to the Breakwater, a distance of three miles, the weather deteriorates, and on reaching the marks for despatching the block, the swell is such that it is extremely dangerous for the pontoon to be manoeuvred into the position previously determined, and to avoid the possibility of being wrecked by blocks in the near vicinity the block is released and by this means reducing the draught of the pontoon.

**Acknowledgments.**

The Author wishes to thank the Civil Engineer-in-Chief, Mr. M. E. Adams, O.B.E., M.I.C.E., for granting permission to present this paper, and to the Civil Engineer Manager, H.M. Dockyard, Devonport, Mr. G. L. Wilson, B.Eng., M.I.C.E., for his help and encouragement.

*Note : A fuller paper on Plymouth Breakwater, for which the author was awarded the South Western Counties Branch Prize for the Session* 1956/7, *is in the Institution's Library. This paper is a documentary history of the breakwater from very early times to the present day and describes the damage it has suffered from nature's elements and man's efforts to combat these terrific forces. It is accompanied by a large number of drawings and photographs.*

# Modern Trends in Prestressed Concrete Pipes

**By I. Alterman, M.Sc.(C.E.), A.M.I.Struct.E.**


### Synopsis

The subject of this paper is the prestressed concrete pipe and its structural elements. Pipes of this description are used as high-pressure conduits, mainly for water supply, with a diameter range from 18 ins. to 108 ins.

The various components of these pipes are designed and constructed with specific objectives in view and to suit conditions which would be difficult to meet by pipes of other description.


The paper is sub-divided as follows :—



## 1. Introduction

MANKIND thrived on water sources throughout the ages. It has always been rather an exception that a community or even an industry relied on water carried from distances, although it was known how to do it for thousands of years. Even nowadays men prefer regions rich in water to arid ones that are richer in other natural resources. The reason is nearly always the high expenditure needed for a large water main and this in turn is due to the fact that a long water-tight structure under pressure requires the best materials and workmanship. Such a structure being a heavy charge on the resources of the community has to be built to last a lifetime. It is obvious that careful planning has to precede every detail of the scheme to ensure good results.

Even the planning itself has to be planned, checked and tested since virtually no " leak " can be tolerated. The methods for design and testing of some large pipe projects are reviewed in the following pages.

## 2. Pipe Design—Generally

### 2.1 *Design Data*

Experience gained in Israel in the design of several hundred miles of concrete pipes, 18 ins.—108 ins. internal diameter, proves the following method suitable :—

(a) Meticulous study of all data pertaining to the design of the pipe, ditch excavation, type of soil, ground-water level, corrosivity of soil and water.

(b) Careful note taken on all river crossings, road and railway data both for crossings and the transportation of heavy pipes to the spot (as well as all other existing structures both above and under the ground).

(c) Contact made with all authorities concerned to find out future plans that may affect the proposed route of the pipe, such as river regulation, road and railway construction, town planning, or land improvement.

(d) Since most of the above items appear over and over again on any large pipe-line project, it is possible to tabulate them, in a stencilled proforma, for the detailed design and final checking, before plans are issued to field construction.

### 2.2 *Methods of Design*

(a) The next step is the structural design and stability computations : the pipe is designed to take both internal pressure water hammer and external loading. These computations determine the details of pipe wall and prestressing wire. Width of excavation, bedding conditions of the pipe and methods of refill for each type of soil result from the external loading data. Once the depth and width of excavation are fixed for every stretch, stability computations call for the design of methods of excavation and strutting, as will be explained further on.

(b) Preliminary sketches and figures are now available for an engineer's estimate of the cost and a time-table for design and field work to be drawn up. These may now be compared with both local and world-wide practice and the comparison can serve as an indication as to whether the design is correct in principle. Quite often this test proves that one or several redesigns are warranted, such as re-routing the line, or even that a substantial change in the hydraulic scheme is called for, e.g. the introduction of one or more boosters, floating reservoirs, and the like.

(c) Before any redesign starts, it is customary to check on remaining special items, such as special precautions against pollution of water in reservoirs, mainly of the open type, drainage systems, or surge tanks. The architectural aspect of all structures left above ground after completion of works is also checked, such as stand-pipes, water towers, reservoirs, pumping stations, boosters, or valve chambers.

### 2.3 *Methods of Checking on the Design*

(a) A large pipe conduit is one of the most expensive structures generally met in engineering work. For comparison, a 108 ins. pressure pipe-line is about 8 times the cost of a standard railway line, on the average mile, whereas the cost of the railway includes culverts, a 150 feet span bridge every ten miles, a full automatic signalling system, and land expropriation.

(b) Modern design of a large pipe calls for careful checking of every detail. After the com-

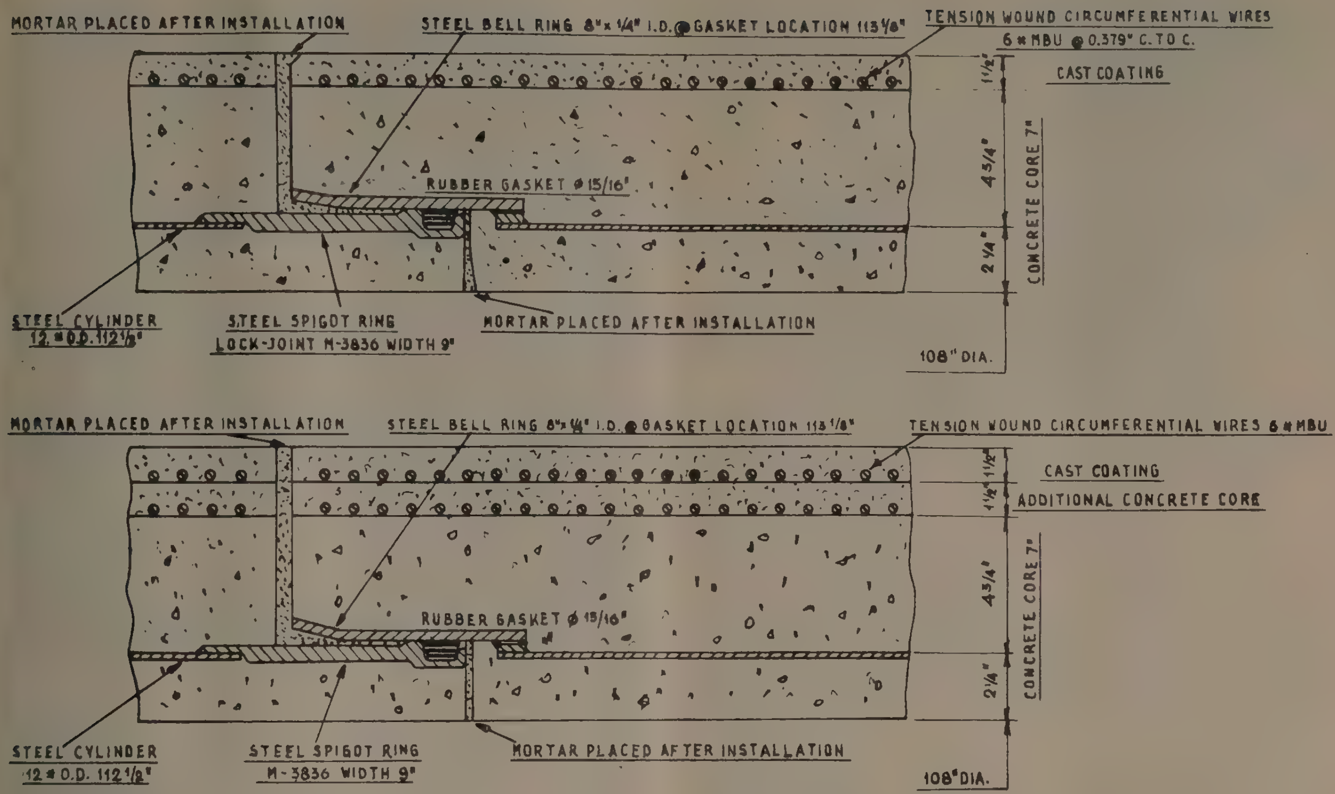


Fig. 1.—Details of 108″ prestressed pipe.

Fig. 2.—Casting yard for pipes 70″ dia. and above. After casting and vibrating the bins are sealed and pipes are steam-cured.

pletion of the preliminary design, further details are worked out, following in reverse the order set out in 2.1 above. This reversal automatically accentuates the predominance of cost over other items, but does not allow the exclusion of any stability features or other basic data.

(c) At last, the final design follows again the original order set out in 2.1. Right at this stage, some doubts may arise due to the many alternative suggestions which had been previously worked out for comparison purposes. Some of the "final" suggestions are either a compromise or a method devised to reduce the price of the scheme: in many instances the compromise represents a novel method as yet untried in practice. In such a case, tests are carried out in order to examine the feasibility of the method which must obviously remain within certain limitations of expenditure and time, although such tests are themselves often extremely costly in comparison with most other structural tests. An example will be given further on of some tests carried out for a 108 in. pipe. It will be mentioned that the cost of these tests amounted to the equivalent of some £60,000 in addition to borings, soil tests, and other investigations which are still to be carried out.

## 3. Structural Description of the Prestressed Pipe

### 3.1 *Pipe Production*

Prestressed concrete pipes as manufactured at the Yuval-Gad Factory in Israel consist of the following basic elements :—

(a) An embedded thin steel cylinder (14 to 16 gauge) is intended to make the pipe perfectly leak-proof. (Each cylinder is tested to some 85 lb/sq. inch).

(b) A bell and a spigot ring, made of special profile, are welded to the ends of the steel cylinder. The bell has a recess to accommodate a rubber ring (see Fig. 1).

(c) High-strength concrete placed on both sides of the steel cylinder, or alternately, centrifugally cast concrete inside the cylinder (see Fig. 2).
The inside of the concrete is extremely smooth in each case (see Fig. 3).

(d) After steam curing in special chambers, the pipes are stripped of their forms. A period of water-curing of 5-7 days makes them ready for prestressing. High tensile steel wire is then wound around the pipe: diameter and spacing of wires are varied for each batch, according to required internal pressure or external load to which the pipe would be subjected. For very high pressures two layers of prestressed wire are used (see Fig. 1).

(e) An external coating is applied to the pipe to protect the steel from corrosive soil or groundwater.

### 3.2 *Pipe Laying*

(a) The pipe is designed to be laid in a narrow trench under considerable earth cover (see Figs. 4 and 5). The trench may either be excavated in advance and the pipe then laid with a crane from the side as in Fig. 4, or preferably the trench may be excavated from

**Fig. 3.—Centrifugating a 66″ internal diameter pipe.**

**Fig. 4.—Large diameter rubber jointed pipe for high pressures (180 lb/sq. in.). Note relatively thin walls due to prestressed method.**

**Fig. 5.—A prestressed 66″ pipe being laid in a curve by opening each joint as specified.**

the front only a few feet ahead of the last pipe and the pipe laid with the help of the same crane excavator.

(b) A rubber gasket is inserted in a groove provided in the spigot and the pipe is pushed home into the smooth bell of the preceding pipe.

(c) The trench bottom need not be prepared to any special form. Each pipe is designed to be laid at a certain invert level and to a certain slope. The latter is adjusted by pushing the spigot to refusal at the top, say, while the bottom is left open a predetermined fraction of an inch. Side deflections are also obtained by opening one side a certain amount. Any required curves are obtained by opening at a certain point on the circumference. For instance, the instructions may read: Open first pipe $\frac{1}{2}$ in. at 5 o'clock, second pipe $\frac{15}{16}$ in. at 2 o'clock, etc.

(d) The soil is backfilled by tamping with pneumatic hammers, somewhat below the springline, then filled with selected soil 1 foot over the top of the pipe and the rest is pushed in with a bulldozer.

## 4. Testing an Experimental 108 in. dia. Pipe Line

An experimental 108 in. pipe, the first of its kind, intended for pressures up to 200 lb/sq. in. was subjected to a special series of tests, to make sure the pipe will perform correctly in the 100-mile long line in which it is intended to be used.

(a) *Hydraulic Test on Single Pipe*

An underground chamber was constructed, into which the pipe was lowered and water was pumped through an internal jacket with the annular space properly sealed at both ends (see Fig. 6). Cracks were inspected with a

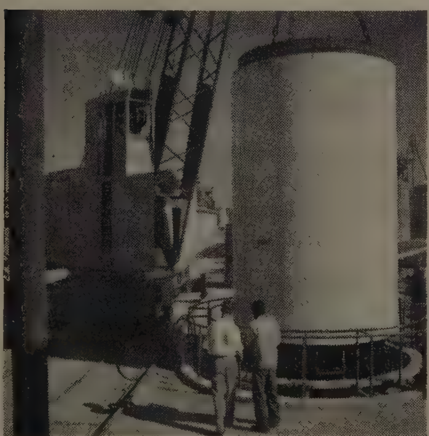

**Fig. 6.—Lowering pipe into underground chamber for pressure testing to destruction.**

special microscope. A few pipes were tested to destruction. A safety factor of 3 was proved in each case. Thus, a pipe designed for 180 lb/sq. in. burst at 570 lb/sq. in. (see Fig. 7). Extensometers attached to the prestressing wire proved that it reached its elastic limit when the pressure rose to 300 per cent of the design working pressure. No leakage whatsoever could be detected at the bell or spigot end, or through the rubber gasket.

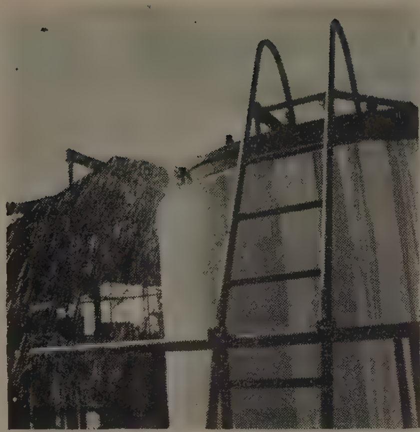

**Fig. 7.—Bursting the 108″ pipe at 570 lb/sq. in.**

(b) *Transportation Tests*

Ten pipes, each weighing about 30 tons, were tilted over, loaded onto a trailer, and transported over rough terrain (see Fig. 8). Not a single crack was caused through this rough handling.

**Fig. 8.—Transporting a 30 ton pipe to site.**

(c) *Test on Pipeline Assembly*

The above ten pipes were laid in a trench and jointed with the aid of a bulldozer (see Fig. 9). The ends of the pipe assembly were closed up hermetically by specially built bulkheads held together by 608 strands of wire stretched between them through the pipeline (see Figs. 10, 11, 12). Intermediate partitions were inserted to hold the wires in line (see Fig. 13). Wires about 160 feet long were threaded through and strained initially with a force of about 250 lbs. each (see Fig. 14). Ends of wires were fixed to each bulkhead with ni-compressed sleeves (see Fig. 15).

(d) The pipes were filled with water and a special pump installation was used to raise the pressure. One bulkhead was free to slide in the bell of the last pipe. In fact the bulkhead moved

Fig. 9.—Pressing home a joint of a 108″ pipe.

Fig. 12.—Outer detail of Bulkhead. Note gadget for reading inner extensometers from the outside.

Fig. 10.—Bulkhead for testing the pipe.

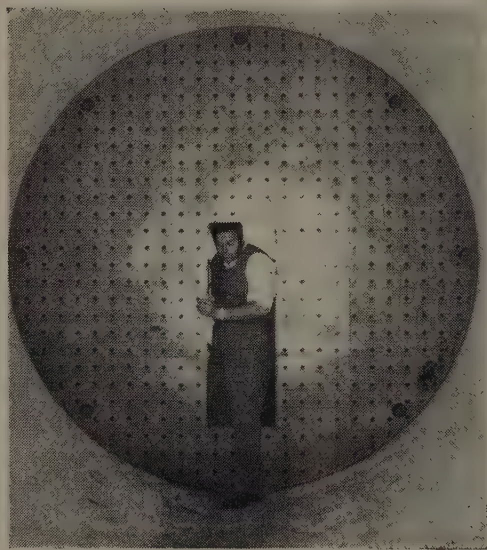

Fig. 13.—Intermediate partitions to hold wires in place—Rectangular opening allows inspection of pipe assembly with wires before and after the test.

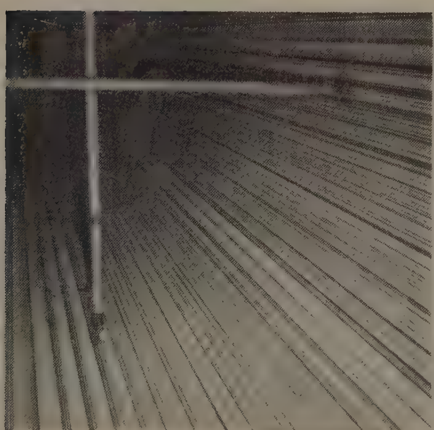

Fig. 11.—Stretching 608 strands of wire between the bulkheads. Note extensometers.

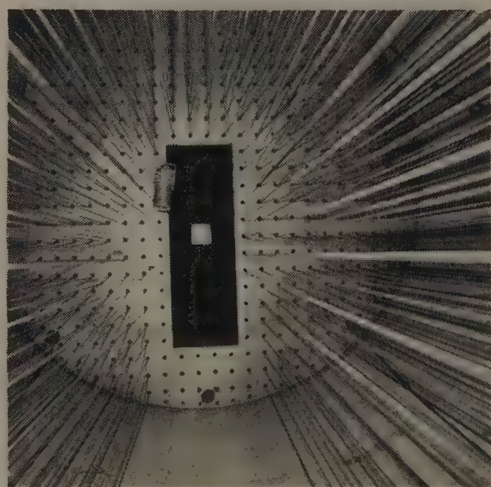

Fig. 14.—Wires strained initially with a force of 250 lb. each. (See simple method of straining in Fig. 10).

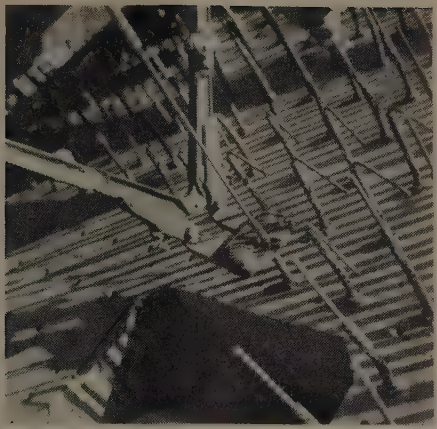

**Fig. 15.—Fixing ends of wires with nicompressed sleeves.**

**Fig. 16.—Subjecting one pipe to three-edge test with two 75 in. jacks.**

like a piston for a distance of 6 ins. when the pressure was built up and the wires stretched. One of the pipes in the line was subjected to a three-edge test both when empty and when under internal pressure. (see Fig. 16).

(e) Internal deflectometers shown in Fig. 11 transmitted their readings through small diameter oil pipes to apparatus fitted outside the bulkhead, as shown in Fig. 12. The apparatus worked to an accuracy of 0.004 in. (1/100 mm.). The external loading reached 215 tons on one single pipe which experienced a deflection up to 0.15 in., while cracks did not exceed 0.003 in. No leakage developed anywhere in the pipe assembly.

### 5. Conclusions and Acknowledgments

It is a major principle of good structural design that full consideration must be given to both the planning and the detailed design of every step, for the best possible results to be achieved in preparation and site work. Unfortunately, this principle is not always observed and its neglect leads to extra expenditure of time and money that could have been avoided. Long-term overall economy calls for careful preliminary work, and the time and money spent on these are more than repaid by consequent savings in "contingencies."

In conclusion, it may be recorded that the whole series of tests on the 108 ins. pipe were designed by Mr. S. Ron, Senior Staff Engineer of Water Planning for Israel Ltd., who also carried out the tests, constructed all apparatus for precise measurement of deflection, devised the special pump arrangement, the three-edge bearing test, and invented and built the movable bulkhead. The author is indebted to him for the photos relating to the test and to Messrs. Water Planning for Israel Ltd. (TAHAL), to the Mekoroth Water Co. Ltd. and to the Yuval-Gad Pipe Manufacturing Co. Ltd. for permission to publish the information relating to their work.

# Book Review

**Analysis of Multistorey Frames,** by Gaspar Kani. Translated from the 5th German Edition by C. J. Hyman. (London: Crosby Lockwood, 1957.) 8 in. × 5¼ in., 113 pp. 40s.

This book describes a method of analysis which puts emphasis on frames with linearly displaceable joints. It uses an iteration procedure which consists of a repetition of the same simple operation. It is claimed that this greatly reduces the probability of a computational error and automatically eliminates errors as the analysis continues. Additionally, the analysis of the multistorey frames considering linear displacements of joints is as simple as for non-translatory joints, and the verification of the final numerical values, from which the end moments are obtained by summation, may be carried out at any time with these values alone.

The book, which includes many numerical examples worked out in detail, and contains useful charts, will be of particular interest to the structural engineer.

# The Use of Electronic Digital Computers in Structural Engineering*

By D. M. Brotton, B.Sc., Ph.D., A.M.I.Struct.E.


### Synopsis

This paper contains a brief description of electronic digital computers and the manner in which programmes are prepared for them. Two examples of their use in structural engineering are included :

(i) The elastic analysis of plane rigid frames first tackled by Livesley.

(ii) The calculation of influence coefficients for an aeroplane wing considered as a cantilever plate.

In conclusion, the use of computers in industry and research and the possibilities of computing services are discussed.


### Description of Electronic Digital Computers

There is an increasing volume of literature on the construction and capabilities of electronic digital computers[1], [2]. Before discussing their application to problems in structural engineering it is necessary first to give a very brief description of these machines and the manner in which a problem is prepared for them.

The electronic computer is a machine designed to carry out arithmetical operations automatically and extremely quickly, for example a simple addition or subtraction instruction may take 180 microseconds and a multiplication instruction 300 microseconds. The basic arithmetical operations of the majority of computers are restricted to addition, subtraction and multiplication although some have automatic division. A number of instructions are also required to initiate operations concerned in machine organisation such as transferring information from one part of the machine to another, controlling the " reading in " of data and the " punching out " of results.

With mechanical electrically operated or hand calculators it is a simple matter say to divide the rotation of a wheel into ten sectors, each one representing a decimal digit. It is therefore convenient to construct mechanical machines to work in ordinary decimal digits. With electronic circuits however, selection of one out of ten would be difficult so a binary system of digits is usually adopted, the presence of a digit being indicated by a charge on the surface of a cathode ray tube, or an impulse in a mercury delay line or a group of magnetic cores. The cathode ray tube, the mercury delay lines and the magnetic cores are three types of fast or working stores which hold the numbers and instructions which are currently being used.

In long calculations the capacity of the working store is not large enough to hold all the data and instructions which are required, so an additional " backing store " is provided. This store usually takes the form of magnetic drums which hold the information as magnetic charges on small segments of the surface. Arithmetical operations cannot of course be carried out in the backing store but information can be transferred to and from the working store in batches.

The operation of the computer is governed by a control unit which holds the control number. The control number specifies the address, that is, the location of the register containing the next instruction which is to be obeyed. In most computers the control number is automatically increased by one when an instruction has been carried out, although means are provided to " jump " i.e. to transfer control to an instruction in an address different from that of the next register.

The arithmetical operations are carried out by an arithmetical unit which comprises an accumulator and multiplier register ; these two fulfil the same functions as the corresponding parts of a hand machine. A certain instruction causes a number in a specified address to be added to a number contained in the accumulator, another instruction will cause a number in the accumulator to be replaced by its product with the number in the multiplier register.

The remaining parts of a computer are those concerned with the " input " of instructions and data and the " output " of results. Information of all kinds which is required by a computer is usually punched on tape or cards. The input unit therefore consists of a tape or card reader which interprets the punched information in accordance with the code being used for the particular type of computer. The reverse process is required for the results produced by the computer. Output may be on punched tape or cards or may be immediately translated and printed out on a teleprinter.

### Programming for an Electronic Computer

The collection and ordering of the sequence of instructions which will perform a particular calculation is known as " programming."

In preparing a programme for a computer it is usual to divide the problem into parts called " Chapters," each chapter being of such a size that all the instructions it comprises, together with all the data which it requires, can be contained in the working store at the same time. The chapters are further subdivided into " Routines," in each of which a particular job will be carried out such as division, obtaining the square root of a number or evaluating an algebraic function. The manner in which the routines are connected together is best illustrated by means of a " Flow Diagram," some of which appear later (Figs. 5 and 9).

Some routines will be required in many programmes and consequently once prepared they are available to all programmers. These are called " Library Routines."

* *Read before the Lancashire and Cheshire Branch of the Institution of Structural Engineers, at Manchester, on 12th February, 1958.*

A considerable amount of labour is involved in preparing a general programme although modern developments in computers tend to make it very much easier. However, before embarking on such a project it is essential to ensure that it will be sufficiently used. Problems involving lengthy calculations but which only require occasional solution may not be worth programming, although the use of "Auto-Code" may be of value in these circumstances. "Auto-Code" consists of a shorthand system of writing a programme and although the machine operations are necessarily carried out less quickly, programming time can be saved.

## The Application of Computers to Structural Problems

For many years the trend of developments in methods of structural analysis has been and still is towards the reduction of tedious arithmetic. This has been accomplished by the introduction of distribution methods, the use of type solutions etc., methods which depend for their efficient use on engineering intuition. Now the computer cannot behave intuitively but it can carry out simple arithmetical operations extremely quickly and accurately; therefore it is necessary to redirect attention to methods which are intrinsically simple and direct but which may involve large amounts of calculation.

It is desirable that the computer is made to do as much of the work as possible. For instance, framework problems could be tackled by manually setting up the equations of joint equilibrium and using the computer to solve them. It is more efficient to give the computer the details of the members and joints and make it set up the equations as well as solve them.

Problems in structural engineering may be divided into two parts, those which involve the setting up and solution of algebraic equations i.e. framework problems and those which involve the solution of differential equations i.e. plates and surface structures. As an example of the first type, the solution of plane rigid jointed frameworks will be considered. The second type will be illustrated by means of a programme to determine the influence coefficients for an aircraft wing of any plan form.

## The Solution of Plane Rigid Frameworks

A programme for the analysis of plane rigid frameworks was first prepared by Dr. R. K. Livesley for operation on the Ferranti Mark 1 machine at the University of Manchester. Several other framework programmes for various computers have been prepared or are now in the course of preparation. The following description of the Livesley programme is in some respects more detailed and less mathematically based than in the report by Livesley and Charlton[3].

The equations used are the slope deflection equations for an inclined member including the effects of axial strains which are usually neglected in structural analysis.

Fig. 1 (i) shows a member inclined at an angle $\alpha$ to the $x$ direction of the general frame co-ordinates with the end reactions due to a unit axial displacement at one end. Figs. 1 (ii) and (iii) show the corresponding reactions due to a unit transverse displacement and a unit angular rotation at the same end. These reactions

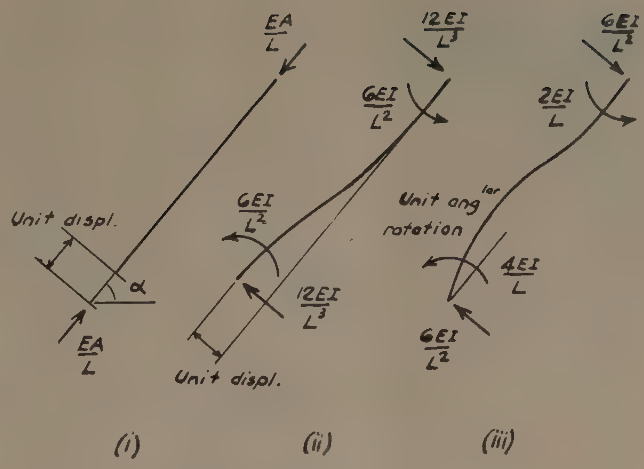


Fig. 1.

due to displacements which relate to individual member co-ordinates can be readily obtained by the elastic theory.

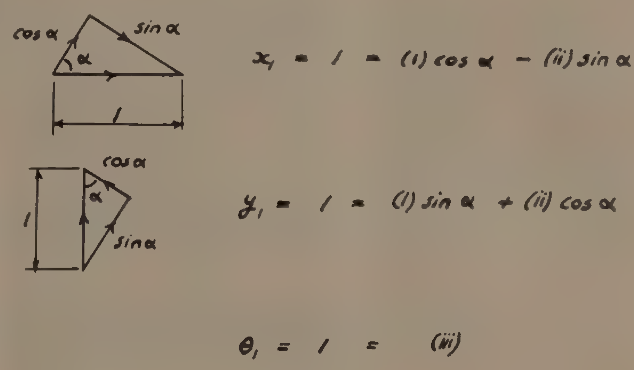


Fig. 2.

In considering joint equilibrium it is necessary to obtain the end reactions due to displacements relative to the frame co-ordinates. It can be seen from Fig. 2 that a unit displacement in the $x$ direction corresponds to an axial displacement $\cos\alpha$ together with a transverse displacement $-\sin\alpha$, similarly the effect of a unit displacement in the $y$ direction was obtained.

Fig. 3 gives the end reactions due to unit displacements in the $x$ and $y$ directions and unit angular rotation at end 1 of the member. These were obtained by substituting the reactions from Fig. 1 in the equations from Fig. 2. By resolving the end reactions shown in Fig. 3 in the $x$ and $y$ directions and considering the end moments, equations can be set up giving the $x$ and $y$ forces and end moment in terms of $x$ and $y$ displacements and end rotation. These equations can be extended in a similar manner to give the same $x$ and $y$ forces and end moment in terms of corresponding displacements at the other end of the member. The six equations which are shown in Fig. 4 result, and since the coefficients relate end reactions to end displacements they are a measure of the stiffness of the member and the group of 36 coefficients is called the stiffness matrix of the member.

$$F_{x_1} = \left\{\frac{EA}{L}\cos^2\alpha + \frac{12EI}{L^3}\sin^2\alpha\right\} x_1 + \left\{\frac{EA}{L} - \frac{12EI}{L^3}\right\}\sin\alpha\cos\alpha\, y_1 + \left\{-\frac{6EI}{L^2}\sin\alpha\right\}\theta_1$$

$$+\left\{-\frac{EA}{L}\cos^2\alpha - \frac{12EI}{L^3}\sin^2\alpha\right\} x_2 + \left\{-\frac{EA}{L} + \frac{12EI}{L^3}\right\}\sin\alpha\cos\alpha\, y_2 + \left\{-\frac{6EI}{L^2}\sin\alpha\right\}\theta_2$$

$$F_{y_1} = \left\{\frac{EA}{L} - \frac{12EI}{L^3}\right\}\sin\alpha\cos\alpha\, x_1 + \left\{\frac{EA}{L}\sin^2\alpha + \frac{12EI}{L^3}\cos^2\alpha\right\} y_1 + \left\{\frac{6EI}{L^2}\cos\alpha\right\}\theta_1$$

$$+\left\{-\frac{EA}{L} + \frac{12EI}{L^3}\right\}\sin\alpha\cos\alpha\, x_2 + \left\{-\frac{EA}{L}\sin^2\alpha - \frac{12EI}{L^3}\cos^2\alpha\right\} y_2 + \left\{\frac{6EI}{L^2}\cos\alpha\right\}\theta_2$$

$$M_1 = \left\{-\frac{6EI}{L^2}\sin\alpha\right\} x_1 + \left\{\frac{6EI}{L^2}\cos\alpha\right\} y_1 + \left\{\frac{4EI}{L}\right\}\theta_1$$

$$+\left\{\frac{6EI}{L^2}\sin\alpha\right\} x_2 + \left\{-\frac{6EI}{L^2}\cos\alpha\right\} y_2 + \left\{\frac{2EI}{L}\right\}\theta_2$$

$$F_{x_2} = \left\{-\frac{EA}{L}\cos^2\alpha - \frac{12EI}{L^3}\sin^2\alpha\right\} x_1 + \left\{-\frac{EA}{L} + \frac{12EI}{L^3}\right\}\sin\alpha\cos\alpha\, y_1 + \left\{\frac{6EI}{L^2}\sin\alpha\right\}\theta_1$$

$$+\left\{\frac{EA}{L}\cos^2\alpha + \frac{12EI}{L^3}\sin^2\alpha\right\} x_2 + \left\{\frac{EA}{L} - \frac{12EI}{L^3}\right\}\sin\alpha\cos\alpha\, y_2 + \left\{\frac{6EI}{L^2}\sin\alpha\right\}\theta_2$$

$$F_{y_2} = \left\{-\frac{EA}{L} + \frac{12EI}{L^3}\right\}\sin\alpha\cos\alpha\, x_1 + \left\{-\frac{EA}{L}\sin^2\alpha - \frac{12EI}{L^3}\cos^2\alpha\right\} y_1 + \left\{-\frac{6EI}{L^2}\cos\alpha\right\}\theta_1$$

$$+\left\{\frac{EA}{L} - \frac{12EI}{L^3}\right\}\sin\alpha\cos\alpha\, x_2 + \left\{\frac{EA}{L}\sin^2\alpha + \frac{12EI}{L^3}\cos^2\alpha\right\} y_2 + \left\{-\frac{6EI}{L^2}\cos\alpha\right\}\theta_2$$

$$M_2 = \left\{-\frac{6EI}{L^2}\sin\alpha\right\} x_1 + \left\{\frac{6EI}{L^2}\cos\alpha\right\} y_1 + \left\{\frac{2EI}{L}\right\}\theta_1$$

$$+\left\{\frac{6EI}{L^2}\sin\alpha\right\} x_2 + \left\{-\frac{6EI}{L^2}\cos\alpha\right\} y_2 + \left\{\frac{4EI}{L}\right\}\theta_2$$

Fig. 4.

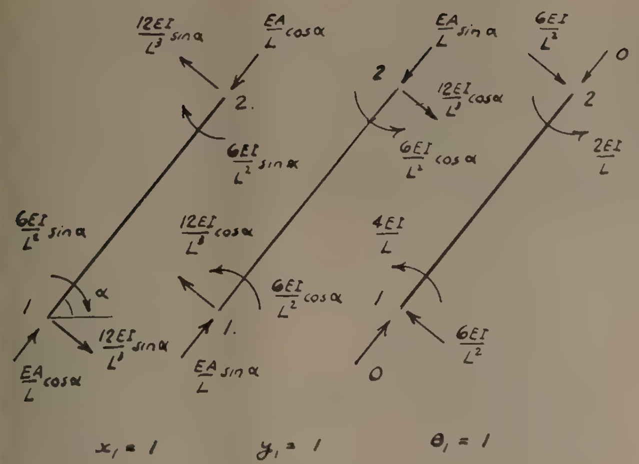

Fig. 3.

The member details which are punched on a " Data Tape " are read in as shown on the Flow Diagram in Fig. 5. The stiffness matrices for each member type are built up and placed in the magnetic store.

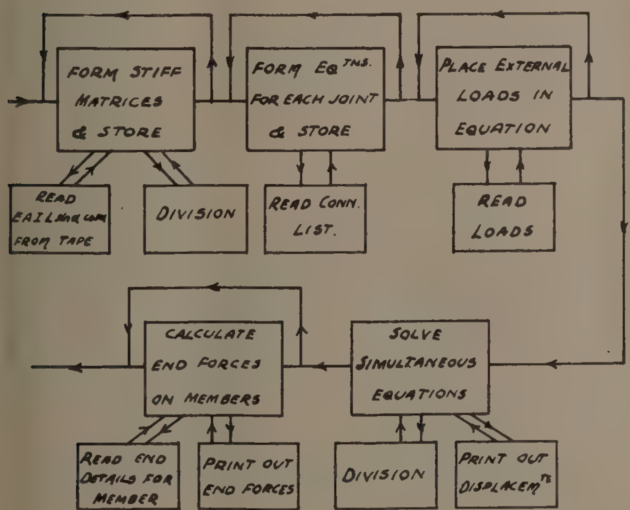


Fig. 5.

In order to set up the three equations of equilibrium at a particular joint, the machine must know the details of the members meeting at the joint. This information, the "connection list," is next read from the data tape. The computer abstracts the rows of the stiffness matrices corresponding to each degree of freedom in turn and groups them together to form the left hand side of an equation. The process is repeated for each joint in the framework, $n$ joints giving rise to $3n$ equations. The external loads which form the right hand sides and complete the equations appear next on the data tape, and are read and placed in their appropriate locations. The final form of the equations for joint 1 is

$$\Sigma F_x = W_x$$
$$\Sigma F_y = W_y$$
$$\Sigma M = W_\theta$$

$W_x$ and $W_y$ are the external loads and $W_\theta$ is the external moment at Joint 1.

Control is now transferred to a routine for the solution of the simultaneous equations and the three displacements at each joint which result are printed out. In the final routine the end displacements are substituted in the original equations to give the axial load, shear and end moments for each member.

The procedure described above is carried out for frames with not more than 10 joints. Larger frames must be divided into sub-frames of not more than 10 joints, each with a number of joints common to two adjacent sub-frames. In this case the first sub-frame cannot be solved completely. The simultaneous equation routine can only eliminate the displacements of the joints not common to the next sub-frame. This will leave reduced equations in terms of the displacements at the common joints and these equations are carried forward to equations of the next sub-frame ; they represent the " inter-action " forces between the two sub-frames. The last sub-frame will be completely soluble and repetition of the process up to the sub-frame before the one which has just been solved will result in a complete solution.

Frameworks with members pinned at both ends can be dealt with by giving the pin-jointed members zero inertia.

### Livesley Stability Programme

As the magnitude of the external loads on a framework is increased the value of the axial loads in the individual members also increases and causes changes in the stiffness of the members. Present day methods of design neglect this change in stiffness which results in increased bending moments and deflections. Using the stability functions tabulated by Livesley and Chandler[4] elastic analysis can be corrected for stability effects.

Livesley prepared an additional programme which when used in conjunction with the one already described will carry out the above process.

The first stage in the calculation is a normal elastic analysis to determine the " non stability " axial loads. This is extended by the calculations of the ratios $\frac{\text{Load}}{\text{Euler Load}}$ for each member. A second analysis with the member stiffness matrices modified by the stability functions corresponding to the previous axial loads gives corrected moments and deflections. The process is convergent and 3 iterations are usually sufficient to reach the correct stability values.

The calculation of the elastic critical load of a two bay pitched roof portal frame with rigid external stanchions and a pinned internal stanchion has been chosen to illustrate the use of the Livesley programme and its extension.

Fig. 6 shows the results of 3 iterations for the two bay cranked rafter subjected to vertical loads of 110,000 lbs. at each joint and a single horizontal disturbing force of 1,000 lbs. applied at the valley joint.

The results tabulated are the $x$ and $y$ deflections in feet and the rotation in radians on one line for each joint in turn. These are followed by a group of 7 figures for each member representing the axial load and shear in lbs. and the bending moment in lbs. feet at End 1 and End 2 of the member and finally the ratio $\frac{\text{Load}}{\text{Euler Load}}$. Comparison of the results from the

**Problem 11 Tape A.2 P = 110,000 lbs.**

+1. Deflections in ft. Rotation radians

| x | y | θ | Joint |
|---|---|---|---|
| +.003588467 | +.008655828 | —.000494796 | / |
| +.007176937 | +0 | +.001979901 | E |
| +.003588467 | —.008670371 | —.000494932 | @ |

| / Load | Shear | Moment (lb.) | |
|---|---|---|---|
| +72294. | +263. | +1165. | End 1 |
| —72294. | —263. | +970. | ,, 2 |
| +0.30309 | | | $\frac{P}{P_E}$ ratio |

| E | | |
|---|---|---|
| +72353. | —119. | +1. |
| —72353. | +119. | —970. |
| +0.30333 | | |

| @ | | |
|---|---|---|
| +71368. | —30. | —104. |
| —71368. | +30. | —820. |
| +0.29920 | | |

| A | | |
|---|---|---|
| +71429. | +166. | +1086. |
| —71429. | —166. | +943. |
| +0.29946 | | |

+1.

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|---|---|---|
| +.008262003 | +.019938507 | —.001497681 |
| +.016524012 | +0 | +.005100059 |
| +.008262003 | —.019953050 | —.001497592 |

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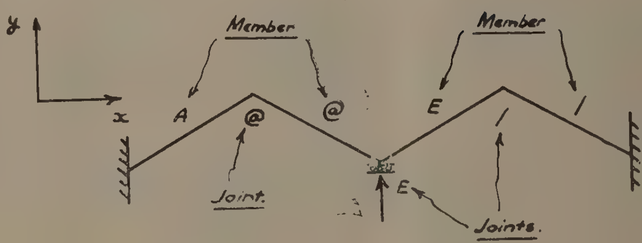


**Fig. 6.**

second and third iterations shows that convergence is complete.

The ratio of $\dfrac{\text{Disturbing Force}}{\text{Horizontal Deflection at Valley Joint}}$ is a measure of the stiffness of the rafter and a series of calculations similar to the above for increasing values of the load parameter shows that the rafter stiffness decreases. Fig. 7 contains a non-dimensional graph of rafter stiffness against load in the rafters and it can be

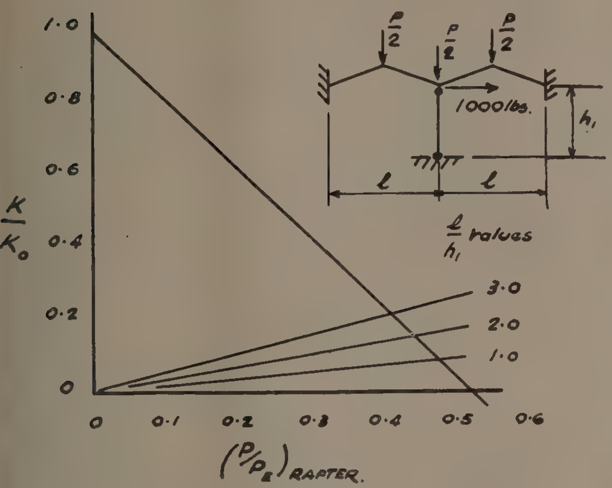


Fig. 7.

seen that zero stiffness is reached for $\left(\dfrac{P}{P_E}\right)_{\text{Rafter}} = 0.53.$ If the framework is completed by a pinned central stanchion the deflection will be increased by the induced horizontal load at the top of the stanchion. The family of straight lines on Fig. 7 represents the induced horizontal forces for a series of different stanchion lengths and the intersections of these lines with the rafter stiffness curve give the zero stiffness values or " elastic sway critical loads " for a series of frames. This example is part of an investigation into the failure loads of multi-bay pitched roof portal frames.

The elastic critical load of a framework is an important parameter on which its failure load depends and its calculation is receiving increasing attention[5].

### Influence Coefficients for an Aircraft Wing

The influence coefficients are the deflections at various points on the wing plan due to the application of a unit load at one point. They are useful in the determination of the strength, stiffness and natural frequencies and modes of vibration of the wing structure. In this solution the wing is considered as a cantilever plate of varying thickness, an assumption which is valid for thin heavy skinned wings[6,7].

The differential equation governing the deflection of a normally loaded plate is

$$\frac{\partial^4 w}{\partial x^4} + 2\,\frac{\partial^4 w}{\partial x^2\,\partial y^2} + \frac{\partial^4 w}{\partial y^4} = \frac{q}{D}$$

where $w$ = plate deflection
$x, y$ = plate co-ordinates
$q$ = surface load intensity
$D$ = flexural rigidity of plate

If the wing plan area is divided into a number of stations, distance $h$ apart, by a square mesh, it can be shown that the above equation can be replaced by the finite difference expression

$$\frac{h^2 Q}{D} = 20\,w_0 - 8\,(w_1 + w_2 + w_3 + w_4) + 2\,(w_5 + w_6 + w_7 + w_8) + (w_9 + w_{10} + w_{11} + w_{12})$$

for internal stations. The suffices refer to the station being considered (station 0) and 12 neighbouring stations shown in Fig. 8. $Q$ is the load applied at station $O$.

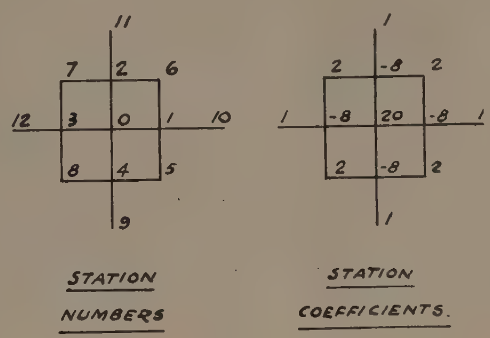


Fig. 8.

For stations on the edge and one mesh length removed from the edge some of the neighbouring stations will be missing ; however, for each missing station a physical edge condition will be available which when expressed in finite difference form can eliminate the unwanted term from the full expression. At a free edge parallel to the $x$ axis the bending moment about the edge $M_x$ must be zero

$$M_x = -D\left(\frac{\partial^2 w}{\partial x^2} + \nu\,\frac{\partial^2 w}{\partial y^2}\right) = 0$$

where $\nu$ is Poisson's Ratio and in finite difference form we have

$$2w_0\,(1+\nu) - (w_1 + w_3) - \nu\,(w_2 + w_4) = 0$$

Finite difference equations can be set up for each station and the solution of these equations will give the influence coefficients for the chosen loading position.

### A General Programme to Calculate the Influence Coefficients

This problem, the theoretical solution to which is given by Dr. D. Williams,[7] was prepared for operation on the DEUCE computers at the Royal Aircraft Establishment, Farnborough. Due to the limitations of storage capacity and the reasonable minimum number of stations required, it was decided to prepare the programme to set up the equations, form the matrix in four parts and punch out each part. Library routines would then perform the matrix operations necessary to produce the influence coefficients for a maximum of 120 stations.

Input to the DEUCE computer takes the form of punched cards and in order to reduce the amount of preliminary work and the volume of initial data required for a particular problem the programme was designed to read in a map of the wing plan form, the location of a station on a 120 × 120 area being

indicated by a digit, i.e. a hole punched in the appropriate position on a card. The only other information necessary is the value of Young's Modulus, Poisson's ratio for the material, the mesh length and the wing thickness at each station. A simplified flow diagram is shown in Fig. 9.

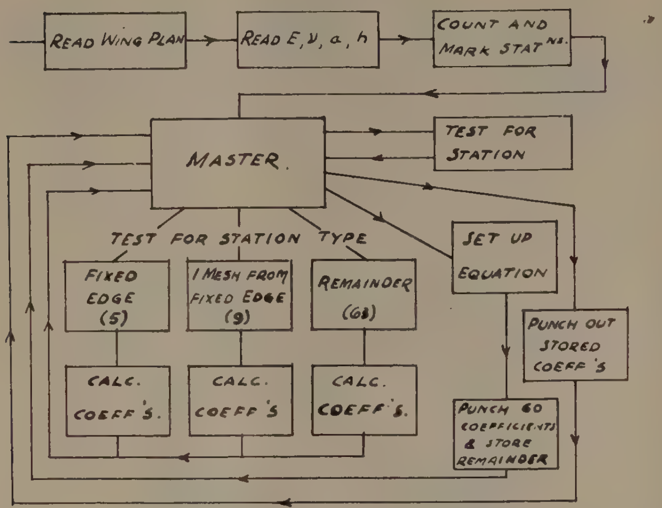

Fig. 9.

The first three routines are concerned with reading in the information and counting and marking the stations, then the master routine which controls the later operations is entered.

As explained earlier, the coefficients of the finite difference expressions for the stations on or near the edge will differ due to the fact that some neighbouring stations are missing. Thus, before we can set up the expression for a particular station we must first determine its type, that is, the pattern of those of the twelve neighbouring stations which exist. There are many different types of station and so in order to reduce the number of tests necessary to determine the station type all the fixed edge stations are dealt with first, followed by the row of stations one mesh length from the fixed edge and finally the remaining stations are dealt with row by row (Fig. 10). Treating

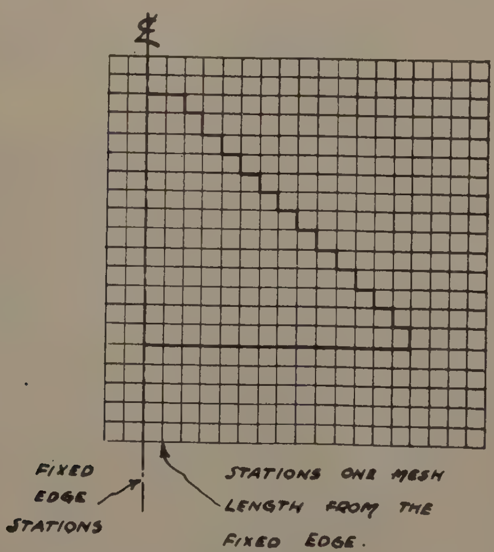

Fig. 10.

them in this way the numbers of possible member types in each of the above groups are 5, 9 and 63 and these require 3, 4 and 6 tests respectively. Once the station type is determined one of a series of parallel routines is entered, each of which calculates and sets up the appropriate coefficients, then returns control to the master routine.

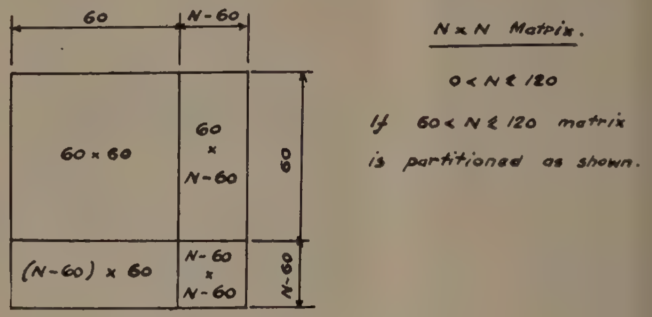

Fig. 11.

The next routine is the one which builds up the equations, placing the previously determined coefficients in their appropriate positions. The matrix is partitioned as shown in Fig. 11 by punching out the first 60 coefficients of each equation and storing the remainder. Finally the stored coefficients are punched out in two groups.

The partitioned matrix can be inverted[8] by means of the available library routines and the product of any load vector and the inverted matrix will then give the deflections corresponding to the particular load.

The detailed coding for this programme has not been carried out at the time of writing so it is not possible to give a worked example.

### Use of Electronic Computers in Industry

It is of interest to study the functions of the various types of personnel who are engaged in the use of computers.

First we have the "Programmers." In the early days of computing they were almost entirely mathematicians by training, but developments which have simplified programming have enabled more and more engineers to prepare programmes. It seems probable that this trend will continue. The programmer must know the code and capabilities of his computer and in addition have some knowledge of numerical methods of analysis.

The "Machine Operator" is an extremely useful member of a computing organisation. His function is the preparation of data tapes and the machine operation for standard programmes. Although obviously an advantage, it is not essential that he should have a detailed knowledge of the code and it is better that his basic training should be in the subject of the programme rather than in computing since he will then be better able to detect errors in the solutions.

Modern computers are extremely reliable; it can reasonably be expected that more than 95% of

scheduled operating time will be useful. However, when they do break down an efficient maintenance engineer must be on the spot to put the machine back into a serviceable condition as soon as possible.

## Computing Services

At the present time, computers are owned and operated by the universities, Government and other research organisations and some large firms in industry. Machine time is available on a commercial basis on quite a number of these computers and at least one offers to put through data tapes for standard programmes and return the results by post. This scheme could be a basis for a computing service to Industry; however, there are better ways of organising a service to structural engineers.

A fully comprehensive service could be provided. The users would be required to send, in an approved form, full details of the structure and the loading. In the case of frameworks, a convenient way of accomplishing this would be with a dimensioned line drawing of the framework together with tables giving the member details (cross-sectional area, moment of inertia, length and inclination relative to some frame co-ordinates of each member) and loading details (values of the components of external loads in the directions of the frame co-ordinates and external bending moments applied at each joint), also the conditions of the supports. The main advantage from the users' point of view would be that they would not need to employ any staff trained in computing. It would appear probable therefore that this type of scheme would be advantageous to the smaller firms whose work would not warrant the employment of full time computing personnel. Sections for the members must initially be arrived at either as a result of previous experience or approximate calculations but it must be remembered that this would have had to be done in any event, the tedious arithmetic of a rigorous analysis is avoided. Difficulties may arise in placing responsibility for the accuracy and correctness of the results produced by such a service but the use of operators with a good structural background and the reliability of modern computing machines should minimise them.

An alternative scheme which could be operated by the larger firms would need the formation of a computing section or department. The staff would prepare programmes or utilise existing programmes for machines on which machine time is commercially available. The main advantage of this scheme is that the firm is in full control of the analysis throughout and any re-analysis necessitated by a change of section can be carried out immediately merely by a small alteration to the data tape.

## Use of Computing Machines in Research

It is convenient to consider research problems in two sections, first "one off" problems such as the calculation of data to compare with model experimental data etc. and secondly problems for which it is desirable to write a general programme.

"One off" problems can be tackled economically by doing a considerable amount of preliminary and possibly final work by hand and using the computer to perform standard arithmetical operations using library routines. Thus, equations can be built up manually and a library routine used to solve them. An alternative would be to write the complete programme in Autocode.

In research it is often necessary to repeat a particular calculation many times with varying initial data in order to determine trends in behaviour resulting from changing the values of one or more parameters. A general programme is extremely useful in these circumstances and there are already several examples of this kind of use where the computer results are treated as results from a "mathematical model."

If full use is made of these valuable machines more complex structures requiring long and tedious hand calculation can be brought within our reach, thus we may be able to tackle 3 dimensional rigid frames, shell roofs etc. with the same facility as we now deal with portal frames.

### Acknowledgments

The author wishes to express his gratitude to Dr. R. K. Livesley for the use of his General Structures programme and thanks the Ministry of Supply for permission to use the Influence Coefficient programme.

The work has been carried out in the Department of Structural Engineering, Manchester College of Science and Technology.

### References

1 Bowden, B. V., (Ed)., "Faster than Thought—A Symposium on Digital Computing Machines." *Pitman.*
2 Livesley, R. K., "An Introduction to Automatic Digital Computers Cambridge University Press.
3 Livesley, R. K. and Charlton, T. M., "The use of a Digital Computer with particular reference to the Analysis of Structures" *Trans. North East Coast Institution of Engineers and Shipbuilders,* Vol. 71, 1954.
4 Livesley, R. K. and Chandler, D. B., "Table of Stiffness and Carry-Over Factors for Structural Members under Axial Load." *Manchester University Press.*
5 Merchant, W., "Frame Instability in the Plastic Range." *British Welding Journal.* Vol. 3 No. 8, Aug. 1956.
6 Williams, D., "Recent Developments in the Structural Approach to Aeroelastic Problems." *Proc. Royal Aeronautical Soc.* June 1954.
7 Williams, D., "A General Method (depending on the aid of a digital computer) for deriving the structural influence coefficients of aeroplane wings," *A.R.C. R & M,* No. 3048, 1956.
8 Frazer, Duncan and Coller, Elementary Matrices.

# Web Buckling and the Design of Web Plates*

## Discussion on the Paper by K. C. Rockey, M.Sc.(Eng.), Ph.D., A.M.I.C.E., A.M.I.Mech.E.

THE CHAIRMAN, in proposing a vote of thanks to the Author, said that the subject of plate girder design, and in particular the web design, had received several waves of interest over the years, some of them very well known and others less known. The most recent wave of interest prior to that of Dr. Rockey was the aeronautical one just before and running into the war, leading among other things to the work of Dr. Leggett. At that time it was obvious that there were still a good many gaps to be filled, even for aeronautical use, let alone for general structural use, and it was very pleasing to see that Dr. Rockey had filled so many of those gaps so well.

There had been an earlier wave of interest before the First World War which led to Professor Lilley's book on the design of plate girders, published about 1910, which was a remarkable effort for its time; but there had been a great advance since then, partly due to the aeronautical effort and also due to Dr. Rockey's work.

The Author had shown pictures of tension field webs and the waves which occurred therein, and the paper mentioned the difficulty of choosing the right spot on a wave to get the peak value of the stress, partly because the waves moved as the load went up. Did Dr. Rockey have any records of the wavelengths and the way in which they decreased as the shear went up? The Chairman added that he had had an interest in this himself some years ago, and he wondered whether Dr. Rockey had been able to get records. If so, it would be of interest to see them published.

Dr. Rockey had made an effort to decide what was a permissible amplitude of buckle. In the paper, he tended to come down in favour of saying that it was primarily a matter of appearance and that for this purpose, so long as the maximum displacement of the web plate was kept to the order of its thickness, it did not show up much and nobody would complain; and he had thought of a way of handling this for design purposes by saying that he would like to keep the shear within $1\frac{1}{2}$ times the critical shear. This seemed to be a very convenient way. Whilst with the rather thin webs that he had used for experimental purposes one thickness of the plate might be a good guide, when dealing with really big plate girders would the Author still be satisfied with that order of movement, or was there not an absolute limit for other reasons? For example, by the time he got to $\frac{1}{2}$ in. plates and thicker, would he really be willing to have $\frac{1}{2}$ in. displacements, or would he say that $\frac{1}{4}$ in. was the most that he could have?

*Read before a meeting of the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on 13th February, 1958. Professor Sir Alfred Pugsley, O.B.E., D.Sc. (Eng.), F.R.S., M.I.Struct.E., M.I.C.E., F.R.Ae.S., (President) in the Chair. Published in "The Structural Engineer" Vol. XXXVI, No. 2, pp. 45-60. (Feb. 1958).

MR. R. E. LANDAU (Associate Member) said that although the paper appeared to be addressed to the Drafting Committees for aluminium and steel structures —the reason for this comment was that its conclusions were in the form of recommendations for the drafting of specifications—Dr. Rockey had generally omitted any comparison between his own proposals and the requirements of existing or draft specifications. In the case of steel girders, one would welcome more comparisons with the draft B.S.153.

Referring to the other end of the chain between theory, experiment and design method, the Author had omitted in some, but not all, cases to compare his valuable experimental results and empirical formulae with theory. Would he give, for example, the comparison between theory and his expressions on page 47 of the paper for plate buckling coefficients and limiting stiffeners?

Mr. Landau had compared some particular cases of Dr. Rockey's proposals with both B.S.153 and other sources, and the results were shown in the following table:

**Comparison of Limiting Stiffness Coefficients**
values of $n$, where $I = nd_ct^3$
single sided stiffeners

| *Source* | $b/d_c = \frac{1}{2}$ | $b/d_c = 1$ |
|---|---|---|
| *Verticals* (Shear Requirement Designs) | | |
| Rockey (Eq. 4) | 3.60 | 1.28 |
| Stein and Fralich | 3.80 | 0.80 |
| Young and Landau (after Timoshenko) | 2.76 | 0.34 |
| B.S.153 Draft | 6.00 | 1.50 |
| *Horizontals at Mid-height* | | |
| (a) *Shear Requirement* | | |
| Rockey (Eq. 7) | 0.26 | 1.04 |
| Young and Landau | 0.08 | 1.48 |
| Adapting Rockey's Eq. 4 | 0.64 | 3.60 |
| (b) *Bending Requirement* | | |
| Massonnet | 0.35 | 0.35 |
| Hampl | — | 0.92 |
| D.I.N. 4114 (German Specn.) | 0.12 | 0.12 |
| (c) B.S. 153 Draft (Shear or Bending) | 1.0 | 1.0 |

Comments on Table:

(i) *Verticals* Dr. Rockey's formula gave good numerical agreement with Stein and Fralich's theory. It would also be noted that the draft B.S.153 figures covered Dr. Rockey's requirement.

(ii) *Horizontals at mid-height.* It might be expected that the limiting stiffness of the horizontals would be approximately the same as for a girder of depth $b$ with vertical stiffeners only at a pitch of $\frac{d_c}{2}$. For this reason the adaptation of Rockey's equation (4) was put in the table, and the discrepancy between the resulting figures and those for equation (7) required some explanation.

For the case of combined bending and shear, Fig. 18 of the paper showed that the best position of a single horizontal stiffener varied from mid-depth to a position near the compression flange. Draft B.S.153, requiring a single horizontal stiffener to be close to the compression flange in all cases, might be criticised on this account. However, having shown where to put the stiffeners, the Author had not made clear how to design them for combined bending and shear, a condition occurring to some extent in all girders and particularly in continuous and cantilever girders.

For central horizontal stiffeners, Dr. Rockey had given equation (7) for shear and equation (27) for bending. For combined stresses, Massonnet[1] had proposed taking the larger of the two requirements, whereas Young and Landau had suggested taking the sum. The German D.I.N. 4114 had a complicated requirement depending on the relative shear and bending stresses. Draft B.S.153 gave the same figure irrespective of the loading condition. Dr. Rockey's comments on this would be welcomed.

Considering vertical stiffeners supporting horizontals, the experimental result quoted on page 47 was most interesting, particularly on account of the simplicity it afforded in design procedure, the design of verticals becoming independent of the horizontals. One wondered, however, whether the test described might have been a special case.

The web panels between verticals were approximately square, and the verticals then had relatively little effect on buckling. If the pitch of the verticals had been halved, a different result might have been obtained. The point could be illustrated by quoting the buckling coefficients for simply supported webs (Fig. 1) if expressed in terms of the depth, as follows:

pitch $= \infty$ : 5.4
pitch = Depth : 9.4
pitch $= \frac{1}{2}$ Depth : $4 \times 6.5 = 26$

Professor B. G. Neal said that in November 1955, a Report on Structural Safety was presented to the Institution by a Committee of which the present President of the Institution was Chairman. In this Report it was pointed out quite clearly that in developing a design method for any type of structure, it was necessary first to consider the safety of the structure in respect of failure at its *ultimate load*, and secondly to consider any other special requirements for the particular type of structure which might determine what was called a *limiting load*. One might therefore have expected the Author, who was investigating a failure problem, to have been concerned with the determination of ultimate or failure loads, and yet his main interest had been with buckling loads.

The buckling loads that the Author had determined were not, of course, the failure loads for this particular type of structure. When a plate girder web buckled, there was merely a redistribution of stress and the loads were carried in another way.

The reason why the Author had been so interested in critical buckling loads was that it was necessary to express actual loads in terms of the critical buckling load to describe the growth of deflections and stresses, so that the critical buckling load had to be known before it was possible to discuss the design of this type of structure.

What the Author had done was to make his girders so free from imperfections that the critical buckling load showed up very clearly in his experiments. This was perhaps the feature which distinguished his work which had been done on plate girders. The outcome had been that the Author was able to predict buckling loads with a high degree of accuracy, but of course actual plate girders were never free from imperfections. Professor Neal wished to hear the Author's views on the effect of imperfections, which certainly played an important part in the design of compression members.

Turning to the possible development of a design method for plate girders, Professor Neal reverted to the Report on Structural Safety. The ultimate load to which the Committee had referred would be some form of complete collapse. In the Author's tests the webs ultimately collapsed by fracture due to high shear stress. The results in the paper showed that when fracture occurred, the shear stress in the web for the particular material used was about 11 tons/sq. in. It was therefore relevant to discuss the value of the working stress to be chosen in order to ensure an adequate margin of safety.

According to the Committee's Report, the appropriate load factor could be determined by considering two groups of factors, firstly those that could affect collapse, and secondly those associated with the seriousness of a collapse if it occurred. In the first group, it was necessary to decide in category A whether, for example, the girder would be manufactured accurately and erected under ideal conditions. For a plate girder bridge or a plate girder forming part of a factory building, it could be assumed that the workmanship would be very good. For category B, which was concerned with loading, it could be stated that for such structures the loading was usually known to a high degree of accuracy. Finally, for category $C$ it could be supposed that with the large body of experimental work available, the collapse condition could be predicted very accurately. On the other hand, such structures were usually statically determinate and there would be little warning of failure. From the tables in the Report, it then appeared that the load factor $X$ to be assigned in respect of this first group of factors affecting collapse was 1.3.

The second group of factors was concerned with the seriousness of collapse should it occur. Category $D$ referred to the danger to human life involved, which for the cases cited would be very serious. Category $E$ referred to the economic consequences of failure, which could be classified as serious. From the tables, the load factor $Y$ in respect of this second group of factors was then found to be 1.5. The overall load factor recommended would therefore be the product of the $X$ and $Y$ factors, which was about 2. Therefore the allowable shear stress in the web should be about 5.5 tons/sq. in. as far as failure was concerned.

In this paper the Author pointed out that current American practice specified a safety factor of 2.4. With the material that he had been using, this would give an allowable stress of less than 4 tons/sq. in. The use of this safety factor, therefore, would be too conservative if the recommendations of the Committee were to be followed.

The other aspect of the Committee's recommendation was to consider whether there was also a limiting load, in other words were there any other factors that would render the structure useless before it collapsed? Here, there was an obvious answer, that the excessive wrinkling of the web might become undesirable before failure occurred. The Author's suggestion was that the wrinkling became excessive when the load was 50 per cent above the buckling load.

The situation was summarised in Fig. 16. In this figure the horizontal line referred to conditions in which the ultimate load condition governed, and the curves referred to conditions in which the limiting load governed. Professor Neal felt that in this figure the ultimate stress was too low, and that it could be raised to about 5.5 tons per sq. in. It would be a pity if the value of the Author's work was partly lost by choosing a load factor which was far too conservative.

MR. K. BOGUSLAWSKI described the paper as a valuable contribution to the design of plate girders. Engineers and designers would find it a valuable guide in understanding the phenomena of buckling.

It seemed, however, that when concerned with the design of deep web girders, especially in bridge construction, the Author had omitted the most important point : i.e., the maximum economical depth of web. All the formulae and diagrams presented in the design paragraph applied only to webs of a maximum depth of approximately 16 ft. Today, there were already a few bridges built which had webs of about 22 ft. depth—e.g., in Germany, the Neuss Dusseldorf Bridge. There was even one bridge of a big girder construction which would shortly be built having webs 26 ft. deep in a box type girder.

In practice, the vertical stiffeners of deep web girders, in addition to buckling, had to resist moments induced by loads from cross girders. This brought to the forefront the loading of web plates beyond buckling loads.

It might be admissible to load webs beyond buckling loads and girders up to, say, 16 ft., but above that depth one would apply it only with reluctance, as relatively little was known about the behaviour of the webs.

On page 56, in paragraph 2 of Section VII (a), dealing with webs, the paper stated that the maximum depth of the buckles could be as much as the thickness of the web plate. Unless this statement was based upon "Test" observations or upon practice, it would appear to be somewhat debatable, especially if deep plate girders with relatively thick webs were being considered.

It might be of interest to state that in the case of a thick plate girder web of a depth of about 16 ft. and a thickness of about $\frac{7}{16}$ in., the lateral deflection of the web was about $\frac{1}{45}$ of the thickness. These measurements were taken recently from one of the bridges designed by Mr. Boguslawski's office in accordance with the German specification D.I.N.4114 and agreed very well with the findings in the paper by Mr. Young to the Institution of Civil Engineers. The size of the panels was approximately 5 ft. 6 in. wide $\times$ 2 ft. 6 in. deep.

Further, still dealing with deep web design, it was found that placing the horizontal stiffeners at 0.2 $d_c$ from the compression flange resulted in a very thick web, because the plate had to be checked against buckling and a buckling coefficient found. The plate had been found to be about 1 in. thick. How, then, should this coefficient be varied to enable a more economical web to be used and also the appropriate horizontal stiffener ?

Again, the use of formula for vertical stiffeners for deep web girders resulted in very heavy and uneconomical sections.

Working in a Continental unit, Mr. Boguslawski had found that in a girder approximately 25 ft. deep, the moment of inertia for a vertical stiffener would require a section 24 $\times$ 7½ joist, which was much too big and it would be much more economical to design a truss girder, instead of a web plate girder.

It was his opinion that the limit in the depth of plate girders should be defined in relation to the data given in the Author's excellent paper.

MR. J. MCHARDY YOUNG (Member) said that having followed Dr. Rockey's work for some years, it was a great pleasure to participate in the discussion. The paper was very timely in view of the present tendency to build more and more comparatively large span bridges as plate girders, with the result, as already mentioned, that there were now plate girders of the order of 20 ft. deep.

In the bibliography, the Author had mentioned the paper by Young and Landau, and in going through Dr. Rockey's paper it was interesting to note the points of similarity. Figs. 11 and 12, for example, dealt with combined stresses, the Author's treatment of which was perfectly correct. He had treated three separate cases and his interaction curves were correct. It was interesting to note that his equation (12) was exactly the same as equation (12) in the paper by Young and Landau, but probably this was merely coincidence.

Mr. Young did, however, cross swords with the Author concerning his reference on page 47 to the effect of stiffening a panel horizontally and its effect on the vertical stiffener. Taking Dr. Rockey's own formula $K = K + A(\gamma)^{\frac{1}{3}}$, it could be shown that the value of $\gamma$ could be reduced as much as 50 per cent with comparatively little reduction in the value of $K$. In Fig. 2, this effect could be seen fairly clearly.

From Fig. 16—the comparison of light alloy panels with and without horizontal stiffeners, it could be seen that the shear value could go up by almost 100 per cent. in some cases for certain values of the aspect ratio. The Author's comments on this point would be welcomed.

In considering the stability of the web itself, a very important point to be borne in mind was any initial eccentricity of the web itself.

Concerning deflections, the paper by Wastlund and Bergman stated that the maximum deflection corresponding to critical load was always less than the plate thickness. It would be interesting to know whether Dr. Rockey agreed with this.

On the same subject, the Author's Fig. 13 showed $\frac{\Delta}{t}$ plotted vertically and $W/W_{cr}$ plotted horizontally, with curves for various values of the area ratio. This group of curves would, however, have been much more useful if instead of plotting the area ratio, the Author had plotted stiffness ratio, because, after all, the area ratio was only a part of the function of $\gamma$.

Mr. Landau had already touched on the vexed and debatable question of stiffeners, both horizontal and vertical, and the comparison between Dr. Rockey's proposals and various other specifications. Mr. Young had himself made comparisons with D.I.N.4114, Table 9. Figs. 6 and 7 were for ratios $\frac{d_c}{4}$ and $\frac{d_c}{5}$ and the Author gave an expression for $\gamma$ in equation (10). As a matter of interest, Mr. Young had worked that out for various values of the aspect ratio from 0.5 to 1.5 and $\delta$ values .05, .1 and .2. That coincided fairly well with the Author's values for $\frac{d_c}{4}$ but for $\frac{d_c}{5}$ the

German values were considerably higher. In view of the graph which had been illustrated by slide—and Dr. Rockey had promised to give a family of curves —it would be interesting to know whether he would like to make any amendment to the formula.

Equation (8) dealt with the case of the stiffeners for panels subject to shear. Again, the German values agreed fairly closely up to the aspect ratio 1.0, but above 1.0 the German values were higher.

Equations (3) and (4) of the paper for vertical stiffeners appeared reasonable and of the right order, but in the section on stiffeners—values and design—the Author had referred to the work by Massonnet. It should, however, be remembered that Massonnet was working with panels with a $d/_t$ ratio ranging up to 4.25 and which would be stable up to collapse loading. Dr. Rockey had suggested that these should be factored, and this was a reasonable recommendation.

Mr. Landau had already dealt with combined shear and bending. One could afford to be a little generous on stiffeners generally, because by employing horizontal plus vertical stiffeners, the thickness of the web was reduced. Even if the area ratio went up, it did so on a reduced thickness value and overall economy was achieved. Mr. Young added the recommendation that where horizontal stiffeners were used, they should be made continuous or properly connected to the vertical stiffeners.

He associated himself with the Author's remarks on page 59 regarding the factor of safety in welded webs subject to combined loading and also on the effect of flexural rigidity of welded flanges on web stability. As the Author had commented on the large number of welded plate girders today and his remarks on the thickness of stiffeners rather seemed to apply to riveted stiffeners, would he specify any recommendation concerning flat welded stiffeners?

Mr. O. A. Kerensky (Member) said that he had read the paper with great interest, anticipating to find at last the answers to one or two vexed problems in the design of webs and stiffeners of plate girders, but he had been somewhat disappointed since more than half the paper was devoted to elastic buckling of webs and stiffeners due to pure shear, pure bending, and the combination of the two, and then halfway through the paper, the Author categorically stated that "in order to achieve a minimum weight design it is necessary to allow webs to operate in the post buckled range" and proceeded to indicate some guiding rules for this type of design. Unfortunately, he did not elaborate the rules sufficiently to make them usable in a design office. In addition, there was no mention of the work of Dr. Brown on the design of post buckled webs which formed the basis of the new rules in the drafts of the revised B.S.153 and 449.[2]

These rules were formulated and checked by a small sub-committee under the chairmanship of Dr. Weck and, as far as he knew, were the first of their kind in the world. They are based on the assumption that in structural practice all webs of plate girders are initially buckled and attempt to give allowable stresses such that the equivalent critical tensile stress (Huber-von Mises-Hencky Theory) in the buckle resulting from coexistent shear, bending and membrane stresses, does not exceed about 90% of the yield stress of the material.

As the Author had avoided drawing comparisons or making critical comments on the proposed rules, Mr. Kerensky had tried to draw his own conclusions and would be grateful if the Author would comment on them. His first conclusion was that the proposed allowable working shear stresses were safe and, if anything, conservative as far as the total carrying capacity of the web was concerned provided that the flanges were sufficiently stiff in the vertical plane. A debt was owed to the Author for pointing out this possible weakness, which appeared to have been missed by everybody else and as far as he knew the Author was the first person to draw attention to it.

From equation 17 on page 56, it would appear that riveted girders with flanges consisting of angles and plates would generally be quite safe, but that in the case of welded girders the danger of premature collapse was real when flange plates were thin and the stiffeners were placed far apart. The results of Author's further tests on the method of calculating the critical inertia of the flange to be used in equation 17 would now be awaited with interest. If the inertia of the plate alone was to be used as implied in the paper the restriction would appear to be extremely drastic.

The Author appeared to be a little critical of the safety factor of 1.1 against yield in the crest of the buckle under combined action of maximum permissible shear and bending and considered that it would be safer to use 1.2. This was a matter of philosophy, but it should be pointed out that with the allowable stresses used up to now in the design of plate girders—i.e. 5-6 tons/sq. in. in shear and 9-10 tons/sq. in. in bending—the Mises-Hencky critical equivalent tensile stress would reach yield when the maximum permissible moment and shear were co-existent. Did not this precedent justify the acceptance of a similar limit in the buckle of the web, which would only be a very local condition?

The second conclusion was that the proposed required inertias of the vertical and horizontal stiffeners were safe and the stiffness of the vertical stiffeners was ample when they were combined with horizontal stiffeners, which was what had been aimed at. When the B.S. rules were drafted the horizontal stiffeners were to be introduced in this country for the first time and experience with their design was not available. The policy, therefore, had been to make the rules as simple as possible and yet amply safe.

Finally, there was the question of whether the first horizontal stiffener should be placed on the neutral axis or at $\frac{1}{5}$th depth from the compression flange. For pure shear, the former was more effective; and for pure bending or a combination of bending and shear, the latter.

As in the majority of cases bending and shear stresses were present together, the B.S.I. Committee had not thought it worth while to cater for the rare cases of pure shear, particularly as in inexperienced hands this recommendation would be unsafe, the danger being that too deep a panel of web subject to shear and bending stress would result.

Mr. Kerensky endorsed the Author's statement that research was needed to perfect the method of design of webs and of horizontal and vertical stiffeners. It was hoped that the Author would be able to find the time and the means to complete this task.

Dr. A. R. Flint, who added his congratulations to the Author on his many years of work which had borne good fruit, wished to comment on the influence that recent research by the Author and others had had on the revision of the Report on the Use of Aluminium Alloys in Buildings. This report had recently been under review by a committee of the Institution and

the problems of web buckling had been studied. The Author's work had been carefully considered and had contributed to improvements in the Report. The design basis adopted was in part conservative when compared with that proposed in the paper for several reasons.

Design shear stresses recommended in the Report were based on the critical (or buckling) stresses or on material shear proof and ultimate strengths. The basic design shear stress adopted for the structural alloy *H* 10 *WP*. was 5.0 tons per sq. in. which corresponded to a load factor of 2.8 on the ultimate and, using the von Mises-Hencky criterion, to about 1.85 on the "yield" value. This latter contrasted with the figure of 2.4 quoted by the Author. These values of load factor were comparable to those of 3.0 and 1.45 for mild steel webs. The argument of a "reserve of strength" beyond the proof stress for such alloys was untenable for *H* 10 *WP*.

While realising that web buckling might not be serious until the critical stress was greatly exceeded it had been considered wiser to continue to employ design stresses based on the theoretical critical value. The load factor had been reduced to 1.5 on the onset of buckling, but the permissible stresses thus obtained were less than a half those recommended by the Author. It was felt that it was unsafe to allow stresses of 1.5 times the critical value in all cases, because of the effects of unknown imperfections and of the non-linear behaviour of most alloys before reaching the 0.1% proof stress. Neither of these was considered by the Author, whose tests were presumably conducted in the linear range with low comparison stresses. It would be of interest to have the Author's comments on these matters, although the question of imperfections has been fully discussed elsewhere.

With regard to the co-efficient, $K$, to be used for design purposes, Dr. Rockey had suggested that partial edge fixity be assumed in the case of riveted plate girders. The stresses proposed in the revised Report were based on this assumption and a load factor of 2.0 for extruded or rolled beams or on simply supported edges and a factor of 1.5 for plate girders, which produced identical working stresses. This avoided the introduction of several formulae but was undoubtedly conservative as a result, except in a few instances.

The Author's work on riveted plate girders with flange angles suggested that flange flexibility was of importance in the post-buckling range. In the case of welded girders there would be very low flange stiffness and it would be implied by the curves in the paper that the critical stress could hardly be exceeded without serious results. Some experimental evidence was obtained from the tests carried out for the B.S.153 Committee at Christchurch. There was no indication of a sudden increase in displacement beyond the critical stress and the measured surface stresses coincided in most cases with those predicted for infinite flange stiffness. This was considered to be due to the effect of flange and web acting together when bending transversely. It would be of interest to know whether the Author had tested idealised girders having no flange stiffness, in which for example, the web might be supported between, but not attached to, tongue plates.

Mr. S. R. Banks (Member) said that while to do so was almost supererogatory he wished to compliment the Author again on the single-minded pursuit of plate-girder phenomena which had now engaged him for some eight years and by which he had placed himself in the forefront of those expert in the subject. Mr. Banks recalled that the present Chairman in his Presidential Address had pointed to the fact that the development of a new material often posed problems the solution of which was applicable to older materials also; and he thought that the Author's investigations, carried out in aluminium, formed an interesting case in point.

He wished to ask three specific questions relating to the comparative behaviour of steel and aluminium in plate girders. The first concerned the aesthetic appearance. Would the fact that the modulus for steel was three times that for aluminium make it likely that buckle-waves in steel girders would be less apparent than in aluminium ones.

Because of the limited equipment available the Author's stiffeners had all been riveted or bolted with one leg flat against the web. There were advantages, as had been shown in one of the introductory lantern slides, in toe-welded stiffeners. These were obvious in steel construction; but with aluminium girders there arose the factor that when a stiffener—or for that matter a flange—was welded there would be a heat-affected zone somewhat weaker than the rest of the web. The degree of edge-support would be altered, and this in turn would affect the web-buckling behaviour. He thought the Author might comment on this.

It is not always easy even in steelwork to decide whether to use a plate-girder or a truss. In fact in many cases the designer would not be able to say precisely why the choice had been made. When the material was aluminium, could the Author cite any factors that would influence the designer to make a different decision than he would with steel?

### Reply to Discussion

Dr. Rockey, in replying, thanked Sir Alfred for his remarks and said that it was both a pleasure and a privilege to address a meeting of the Institution. Sir Alfred had enquired whether he possessed any data regarding the change in wavelength of the buckles with increase in load. He had in his records, contour plots of the buckled webs of several dozen girders and shear panels and in view of the President's comments he would most certainly prepare a report dealing with the changes which occur in the buckle patterns with changing load. He had found that the wavelength of the buckles varied in a manner very similar to that noted by both the President[3] and Dr. Leggett[4]. He had tended to use the absolute depth of the buckles rather than the wavelength in his work because he considered that the latter was not so well defined and therefore involved more personal judgment.

With regard to the appearance of the buckled webs, the survey which had been conducted at Swansea had involved an examination of the appearance of the webs of rather small girders. This meant that an observer was able to cover the whole of the web in a single glance and he considered that this was a more stringent test than would be involved in practice where, because of the size of the members, only a portion of the web could be examined in a single glance. He was therefore happy, as far as aesthetic considerations were concerned, for lateral deflections of up to the web thickness to occur, even when large girders were involved.

In reply to Mr. Landau, Dr. Rockey said that he too would have liked to have been able to have included in the paper more detailed comparison between his recommendations and those contained in current

specifications. However, to have done so in the present paper would have involved the omission of some of the background data, and rightly or wrongly, he had finally decided on the inclusion of the latter. However, most of his earlier papers did include such comparisons, many of which were not very favourable to current specifications here and abroad. He had hoped, as indeed as had happened, that the discussion would bring to the front many of the points of difference between his recommendations and those of other people.

With reference to the expressions contained in equations (2), (3) (4) and (5), these empirical relationships were very similar in form to the theoretical relationships obtained by Stein and Fralich. There were however, differences in magnitude between the empirical curves and the theoretical curves, because the former were obtained from tests on webs which were clamped at their edges, whilst the theory was for the case of pin jointed edges. However, allowing for this, and the fact that the theory did not distinguish between the behaviour of single and double sided stiffeners, the agreement was very good. In an earlier paper,[5] Dr. Rockey had in fact included curves which showed the comparison between the theoretical and empirical curves.

Mr. Landau had also questioned why the inertia which a central horizontal stiffener should possess according to equation (7) is different from that which is obtained by adapting equation (4). The reason for this was due to the fact that for a web subjected to shear, the buckle formation in the longitudinal direction is different from that in the transverse direction. Stein and Fralich[6] in their excellent paper—dealing with the behaviour of webs subjected to shear and reinforced by vertical stiffeners, showed that the relationship between the shear buckling coefficient $K$ and the parameter $\gamma \left(=\frac{EI}{Dd}\right)$ varies considerably with different wave patterns.

In view of this, it was important that engineers should note that the formula which is given in the German Specification B.I.N. 4114 for the design of central horizontal stiffeners on webs subjected to shear has been obtained by simply adapting the formula for vertical stiffeners which is contained in the specification.

Dr. Rockey thanked Mr. Landau for drawing attention to the fact that he had not discussed the requirement for stiffener inertia when both shear and bending stresses were present. There was only limited experimental evidence on this point and he personally subscribed to the view expressed by Professor Massonnet; that you determined the inertia required for the case of pure shear and pure bending respectively and used the larger of the values. It was clear however, that it was desirable that further experimental evidence be obtained to give guidance on this point.

The rules for stiffener inertia contained in the German specification D.I.N. 4114 should at all times be used with caution. It had been clearly established that the formulae contained in the specification for the design of vertical stiffeners on webs subjected to shear provide very low values of $\gamma$, for example considering the two cases when the vertical stiffeners are spaced at $d$ and $0.5d$, the D.I.N. 4114 formulae provide values of $\gamma$ which are only 0.09 and 0.30 respectively of the values required by equation (3). In addition Dr. Rockey had shown[7] that for the case of a web subjected to shear and reinforced by vertical stiffeners and a central horizontal stiffener, the interaction curves contained in D.I.N. 4114 provide unrealistic values for stiffener inertia.

The laws for the design of horizontal stiffeners on webs subjected to bending were more reliable being based upon the theoretical papers by Massonnet and Dubas. It was recommended however, that the multiplying factors as proposed in section (*VI b*) be employed.

The empirical relationships given in equations (6), (7) and (8) were obtained for the special case when the vertical stiffeners were single sided and had a stiffness equal to that required by equation 4. The test programme from which these laws were obtained involved 143 different plate-stiffener combinations and eight different values of the aspect ratio $b/d_c$, covering the range 0.4 to 1.2.

Dr. Rockey in reply to Professor Neal, said that he was glad to have the opportunity of expressing his appreciation of Professor Neal's constant interest in his work. The discussions which he had had with Professor Neal since he had been Professor of Civil Engineering at Swansea had been both stimulating and profitable. Professor Neal had in a clear and concise manner examined the question of suitable load factors for normal plate girders in the light of the Institution's Report on Structural Safety and the experimental evidence obtained from tests on plate girders by the writer and others. He was in full agreement with what Professor Neal had said and supported his suggestion that for the high strength alloys, the allowable shear stress should be 5.5 tons/in$^2$. By employing this more realistic stress, it would be possible to develop a much more economical design procedure.

Professor Neal had enquired about the effect of initial imperfections in a web. As soon as a web possessing an initial deformation is loaded, it will deflect laterally and the bending stresses and membrane stresses associated with buckling will occur at loads much lower than the buckling load. The greater the initial deformation, the larger will be the gain of deflection and the greater the bending and membrane stresses. It was because most welded girders have initial imperfections that he had suggested increasing the factor of safety against initial yield from 1.1 to 1.2.

Dr. Rockey thanked Mr. Boguslawski for his comments. He was most interested to learn of the developments in plate girder design which were taking place on the Continent. The statement on Page 56, paragraph 2 of Section VII to which he referred, was based on the empirical curve given in figures 13 which was determined experimentally from tests on several dozen girders.

Mr. Boguslawski had mentioned that a web deflection of only $\frac{1}{45}$ of the web thickness had occurred in a given girder. For the case quoted the operating stress would have been much lower than the buckling stress and therefore the lateral deflections of the web would have been expected to have been small. When dealing with very deep plate girders it would be necessary to employ several sets of horizontal stiffeners, especially when both shear and bending stresses were acting. This was necessary in order to keep the thickness of the web below 1 in., which is desirable since the inertia required of the stiffeners varies directly as the third power of the web thickness.

Dr. Rockey replying to Mr. Young said that on

examining figure 2, it would be noted that a reduction of $\gamma$ to 50 per cent of $\gamma_{LV}$ involved a reduction in the value of $K$ of some 15 per cent. The experimental results obtained when the size of the vertical stiffeners was varied whilst keeping the rigidity of the horizontal stiffeners constant at a value corresponding to $1.17\gamma_{LH}$, had shown that the rigidity of the vertical stiffeners could be reduced to 66 per cent of the value $\gamma_{LV}$ given by equation (4) before any significant change in the value of $K$ was obtained and in fact had to be reduced to approximately $0.33\gamma_{LV}$ before the value of $K$ was reduced by 15 per cent. The test had shown that vertical stiffeners designed according to equation (4) would act very effectively when used in conjunction with horizontal stiffeners and the empirical formulae given in equation (6), (7) and (8) were derived from tests where the vertical stiffeners had rigidity equal to that required by equation (4).

When a web is loaded in shear, a great increase in $K$ can be obtained by employing a horizontal stiffener at mid-depth in conjunction with vertical stiffeners. For example, if only vertical stiffeners are used and are placed at a distance $d_c$ apart to form a square panel, then if it is assumed that all edges are simply supported, the critical shear coefficient $K$ has a value of 9.34. If however, an effective horizontal stiffener is placed at mid-depth, the square panel is divided into two panels, the $K$ value for each being 6.34 when used with respect to the reduced depth/thickness ratio of $d_{c/2t}$. Therefore with respect to the full $d_c/t$ ratio, $K$ has a value of 25.4, an increase of 170 per cent over that value obtained when only vertical stiffeners employed. Equation (8) provides values of $K$ which lie between the theoretical values obtained when the panels are assumed to be either simply supported or clamped on all edges the empirical values approaching the simply supported edge values with decreasing values of the aspect ratio $b/d_c$.

Mr. Young had quoted Wastlund and Bergman as reporting that at the critical load, the web deflections they obtained were never greater than the thickness of web. The lateral deflections of the web which occur at a load corresponding to the theoretical buckling load will depend upon the initial deformation in the web. If the web is initially plane then the web deflection will be very small, but if the initial web deformations are large these deformations will be increased considerably before the theoretical buckling load is reached, and could well exceed the plate thickness.

The theoretical curves shown in figure 13 were derived assuming that the stiffeners were non-deflecting, that is assuming they had infinite bending rigidity. The experimental curve was obtained from tests where the vertical stiffeners had an inertia in excess of $EI_{LV}$.

Mr. Young had compared the formulae for the design of stiffeners given in the present paper with those contained in D.I.N. 4114 and had noted certain differences. For a web subjected to pure bending the specification D.I.N. 4114 provides formulae for the design of horizontal stiffeners when these are placed at either $0.2d_c$ and $0.25d_c$ from the compression flange. The formulae for the $0.25d_c$ position gives values equal to Massonnet's theoretical values, but the formula for the $0.2d_c$ provides some 20 per cent greater than Dubas' theoretical values. The formulae in the present paper were based on the theoretical values.

With regard to Mr. Young's question regarding the thickness stiffeners should possess when these are toe welded to the webplates, he would suggest that the web of any outstanding leg should not have a thickness less than one-twelfth of the outstand.

Dr. Rockey strongly supported Mr. Young's view that when horizontal stiffeners are used in conjunction with vertical stiffeners these should be made continuous or properly connected to the vertical stiffeners. He had much appreciated Mr. Young's comments in view of his wide experience in the design of long span plate girder bridges.

In reply to Mr. Kerensky, Dr. Rockey said that in preparing the paper he had endeavoured to present a fair survey of the existing state of knowledge appertaining to the buckling of the webs and their subsequent behaviour and also to present the basis of a design procedure. He personally considered that the proposed design procedure could be used in a design office. He would agree that it was not so streamlined as the proposed B.S. 153 but it was a more general treatment and permitted the designer greater flexibility.

The procedure contained in the proposed B.S. 153 was a major advance on that contained in the existing B.S. 153 or B.S. 449 specifications and Mr. Kerensky and his co-authors Dr. Flint and Dr. Brown were to be congratulated on the result of their efforts.

Mr. Kerensky had mentioned that no reference had been made to Dr. Brown's work, the reason for this was simply that it was not possible to refer to all previous work on plate girders, to do so would have meant that the bibliography would have been several times its present size.

The paper only quoted those researches which form the basis of the proposed design procedure.

Mr. Kerensky had stated that the paper contained no comparison of the rules contained in the draft B.S. 153. The paper had, as far as the post buckled behaviour of webs subjected to shear was concerned shown that Leggett's theoretical values and the experimental data were in reasonably close agreement, whilst Bergman's theoretical values underestimated the effect of buckling. Bergman himself recognized this, as indicated by the statement quoted from his book which is on page 54. It is therefore somewhat disturbing to note that Mr. Kerensky and his co-authors when developing their design procedure had used Bergman's theoretical relationships, without the correcting factor as suggested by Bergman himself. This meant that the actual maximum stresses occurring at the crests of waves would be greater than was assumed in the development of the B.S. 153.

In fact it could be shown that at the design stresses, parts of the web are loaded up to or beyond the yield stress of the material. For example, the proposed B.S. 153 permits a shear stress of 4.27 ton/in$^2$. and a bending stress of 9.0 ton/in$^2$. to act together when the $d/t$ ratio is 180 : 1 and the vertical stiffeners are spaced at $1.5d$. Assuming simply supported edges, and most investigators are of the opinion that with the typical flange to web connection used in welded construction the web receives only a pin jointed support, it can be shown that for these design stresses the web is operating at 1.91 times its buckling load. If the web was initially plane, the maximum web deflections would therefore be approximately 1.7 times the plate thickness. In addition, on applying the empirical relationship given in figure 14, it will be found that for this combined loading of shear and bending the yield stress is exceeded at the crest of

the waves. Mr. Kerensky had stated that the draft B.S. 153 is based on the assumption that all webs are initially deformed. If this is so, then in the case under consideration the yield stress would be exceeded at loads well below the design stress and this would result in an even larger region of the web yielding than would occur in a plane web. It could be argued however that the flange does supply a certain amount of rotational restrain to the web and that the assumption of simply supported edges is conservative. This reserve of 'safety' is, in his opinion, more than offset by the effects of initial imperfection and the flexible nature of the customary welded flange. It could therefore be claimed that the stresses in draft B.S. 153 are a little too high, this being the speaker's personal opinion. He was however, fully aware of the magnitude of the Committee's task and considered that members of the structural profession owed much to them for their efforts.

The design laws for both vertical and horizontal stiffeners, whilst not in the form that Dr. Rockey would like, would be adequate. Mr. Kerensky had stated that for a loading of combined shear and bending, the best position for a horizontal stiffener was at $\frac{1}{5}$th depth from the compression flange. This was not strictly true since the actual position would depend upon the stress ratio $T/\sigma$ as indicated in figure 18.

Dr. Rockey in reply to Dr. Flint thanked him for his kind remarks. He was very pleased to learn that in the Institution's report on the Use of Aluminium Alloys in Buildings, the basic design shear stress for the structural alloy H10WP was 5.0 tons/in$^2$. this being much higher than that permitted at present and close to the value suggested by Professor Neal in the course of the discussion.

When preparing design specifications for Aluminium alloys, he thought that the ideal way was to follow the current American practice and to limit the design specifications to certain classes of alloys. The proposals he had made were in fact for the high strength alloys such as H10WP or H30WP. Some of the other aluminium alloys, such as NS5, had a lower ratio of 0.1 per cent proof stress/ultimate stress and this meant that if the 0.1 per cent proof stress was not to be exceeded under operating conditions, the plate girder would have an excessive load factor based on collapse. If it was decided to operate beyond the 0.1 per cent proof stress, then the determination of the critical stresses would be more difficult and only approximate values could be determined, especially when combined shear and bending occurred

The problem of flange flexibility was still being considered. Dr. Rockey would like to correct the statement made by Mr. Kerensky in which he suggested that Dr. Rockey had been the first to note the problem, this was not true, it had in fact been considered by aeronautical engineers for some time, the most notable paper on the subject as far as tension field webs were concerned being that by Leggett and Hopkins.[8] He thought that Dr. Flint's suggestion that a test be conducted in which the flange members only guided the web and were not attached to it was a good one and he would most certainly carry out such a test in the near future.

In reply to Mr. Banks, Dr. Rockey said that the fact that the modulus of aluminium was only one third that of steel meant that the design of aluminium girders had to be based upon a full understanding of the buckling of stiffened plates. It was now possible to design an aluminium plate girder so that the full strength of the material could be utilized, Because of the lower modulus, it was necessary to employ smaller panels than with steel girders, because unless this was done it would be found that the aesthetic requirement became the limiting one not the strength requirement.

Dr. Rockey had not tested any aluminium alloy girder of welded construction, in which, because of the heat treatment received during the welding process, there were areas of web adjacent to the welds which were weaker than the rest of the web. He would have thought that in such cases it would be necessary to assume that the panels received only a simple pin jointed support.

In reply to Mr. Banks' last question, Dr. Rockey thought that because the modulus of aluminium was only one third of steel, the design of column members in trusses became considerably more difficult and for that reason he considered that the plate girder would generally be preferred to the truss when using this material. There was a discussion on the relative merits of trusses and plate girder construction in Technical Notes 2661 and 2662 of the National Advisory Committee of Aeronautics, which might be of interest to Mr. Banks.

THE CHAIRMAN, in concluding the discussion, said that some speakers had referred to the recorded failure of the flange of one of the test plate girders by inward bending when the webs were taken into the post buckling stage. This mode of failure had been foreseen by H. Wagner when developing his early tension field theory in Berlin. He foresaw also that this feature in the behaviour of a plate girder affected the economic choice between a plate girder and a lattice girder. The work by Wagner was published in 1929.

### References

1. *Massonnet, C. Correspondence on paper by Young and Landau. Proc. Inst. C.E., Part I, January,* 1956.

2. *"The Basis for Design of Beams and Plate Girders in the Revised B.S.*153*" by Kerensky, Flint and Brown— Proceed. I.C.E. Part III, Vol.* 5, *August,* 1956.

3. *A. G. Pugsley "On a Strut in a Non-Linear Medium and the Waves in a tension field Shear Web" Symposium on "Engineering Structures" Colston Research Society, Bristol,* 1949. *Publishers, Butterworths.*

4. (*Reference* 26 *present paper*). *D. M. A. Leggett. "The Stresses in a Flat Panel under Shear when the Buckling Load has been exceeded." H. M. Stationery Office R. and M. No.* 2430.

5. (*Reference* 6 *Present Paper*). *Rockey K. C. "The Design of Intermediate Vertical Stiffeners on Web Plates subjected to Shear." Aeronautical Quarterly Vol. VII, p.* 275-296. *November,* 1956.

6. (*Reference* 4 *Present Paper*). *Stein, M., and Fralich, R. W., "Critical Shear Stress of Infinitely Long Simply Supported Plate with Transverse Stiffeners," N.A.P.A. Technical Note* 1851. *April,* 1949.

7. (*Reference* 7 *Present Paper*). *Rockey, K. C., "Shear Buckling of a Web reinforced by a Vertical Stiffener and a Central Horizontal Stiffener." Vol.* 17. *Publication I.A.B.S.E.* 1958.

8. *D.M.A. Leggett and H. G. Hopkins. "The Effect of Flange Stiffness on the Stresses in a Plate Web Span under Shear." H. M. Stationery Office. R. and M. No.* 2434.

### *ERRATA*

*Page* 51. *Fig.* 9. *The vertical scale of Fig.* 9 *represents values of the coefficient K and this letter K has been omitted.*

*Page* 59. *Equation* 27 *should read* $I=0.35t^3d_c$.

# Institution Notices and Proceedings

## PRESIDENTIAL ADDRESS

The Presidential Address for the Session 1958-59 will be given by Mr. Gordon S. McDonald, M.I.Struct.E. M.I.C.E., M.I.Mun.E., at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 2nd October, 1958, at 6 p.m.

## FORTHCOMING MEETING

*Thursday, October 23rd*, 1958

At 11, Upper Belgrave Street, London, S.W.1. Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when a paper on " Aluminium and Tubular Steel applied to the Construction of some recent Aircraft Hangars " will be given by Mr. L. E. Ward, M.I.Struct.E.

Members wishing to bring guests to the Ordinary Meeting are requested to apply to the Secretary for tickets of admission.

## FIFTIETH ANNIVERSARY CONFERENCE

The Fiftieth Anniversary Conference will be held in London on the 7th to 10th October, 1958. Members wishing to take part in the Conference who have not yet returned their completed Registration Forms are invited to do so as soon as possible.

Official delegates from technical bodies in many parts of the world are attending the Conference. This will commence on Tuesday evening, 7th October, with a Reception at Londonderry House, Park Lane, by Mr. G. S. McDonald, who will be installed as President of the Institution on the 2nd October, 1958. The Technical Sessions will be held on the mornings of Wednesday, Thursday and Friday, 8th-10th October. Tours of engineering interest will take place. Alternative tours and visits of general interest are available and the programme includes a variety of social functions and entertainments. The Conference will terminate with a Banquet and Dance at the Dorchester, Park Lane, on Friday, 10th October.

## GOLD MEDAL OF THE INSTITUTION

The Council have awarded the Institution Gold Medal to Professor Hardy Cross for his outstanding contribution to the science and art of structural engineering during the past forty years.

## EXAMINATIONS—JANUARY, 1959.

The Examinations of the Institution will be held in the United Kingdom and overseas on Tuesday and Wednesday, January 6th and 7th, 1959 (Graduateship), and Thursday and Friday, January 8th and 9th, 1959 (Associate-Membership).

## SPECIAL JUBILEE ISSUE OF " THE STRUCTURAL ENGINEER "

Copies of the Special Jubilee Issue of " The Structural Engineer " which was published on the 21st July to mark the Fiftieth Anniversary of the Institution are available, price 10s. 6d. (by post, 12s. 0d.), on application to the Institution of Structural Engineers, 11, Upper Belgrave Street, London, S.W.1.

## THE INSTITUTION OF STRUCTURAL ENGINEERS

## APPOINTMENT OF DEPUTY SECRETARY

Applications are invited for the appointment of Deputy Secretary of the Institution of Structural Engineers, with a view to eventual appointment as Secretary.

Applicants should preferably be corporate members of a Chartered Engineering Institution, have appropriate administrative experience and be between the age of 35 and 50.

The commencing salary will be from £1,800 to £2,000 per annum according to experience.

The closing date for the receipt of applications is the 24th October, 1958.

For further details apply to the Secretary of the Institution, 11, Upper Belgrave Street, London, S.W.1., the envelope to be marked " Deputy Secretary."

## LONDON GRADUATES' AND STUDENTS' SECTION

The opening meeting of the Session will be held at 11, Upper Belgrave Street, London, S.W.1. on Tuesday, 14th October, when Mr. Johnson Marshall, A.R.I.B.A., A.M.T.P.I., will give a talk on " Design of Cities."

*Hon. Secretary* : R. M. Amodia, B.E., 21, Wetherby Gardens, London, S.W.5.

## BRANCH NOTICES

### MIDLAND COUNTIES BRANCH

The Annual Dinner and Ladies' Evening will be held at the Botanical Gardens, Birmingham, on Saturday, 18th October.

A Meeting of the Branch will be held at the James Watt Memorial Institute, Great Charles Street, Birmingham, on Friday, 24th October, at 6.30 p.m., when the Chairman's address will be given by Mr. B. C. Britton, M.I.Struct.E., F.R.I.C.S. Tea will be served from 5.45 p.m.

*Hon. Secretary* : John R. Chaffer, M.I.Struct.E., 107, Jockey Road, Sutton Coldfield, Warwickshire.

### GRADUATES' AND STUDENTS' SECTION

The first meeting of the Session will be held at the Birmingham Exchange and Engineering Centre, Stephenson Place, Birmingham on Friday, 31st October, 1958, at 6.30 p.m., to hear Mr. Ove Arup, C.B.E., M.Ing.F., M.I.Struct.E., M.I.C.E., on " Future Trends in Structural Engineering." Tea will be served from 6 p.m.

*Hon. Secretary* : F. A. Butterworth, " Roscrea," Tansey Green, Pensnett, Brierley Hill, Staffs.

## LANCASHIRE AND CHESHIRE BRANCH

# CELEBRATION OF THE FIFTIETH ANNIVERSARY OF THE INSTITUTION

**27 OCTOBER 1958 1 NOVEMBER**

### INAUGURAL MEETING

28th OCTOBER, 1958, 7.30 p.m.

A hot-pot supper will be held in the Refectory of the College of Science and Technology, Manchester at which the President will install the Branch Chairman, Dr. D. D. Matthews.

### SYMPOSIUM ON STRUCTURAL ENGINEERING

WEDNESDAY, 29th OCTOBER, 1958

Papers will be presented by
PROFESSOR HARDY CROSS SIR RICHARD SOUTHWELL
PROFESSOR SIR ALFRED PUGSLEY
with discussion by
PROFESSORS J. A. L. MATHESON, W. MERCHANT and J. B. OWEN at THE COLLEGE OF SCIENCE AND TECHNOLOGY, MANCHESTER and at the UNIVERSITY OF MANCHESTER

During the Symposium Professor Cross will be presented with the Gold Medal of the Institution in recognition of his contributions to structural engineering.

ADMISSION LIMITED TO TICKET HOLDERS — SEE BELOW

### CIVIC RECEPTION

The Lord Mayor and Corporation of the City of Manchester graciously invite members of the Branch to a Civic Reception in the Town Hall, on Wednesday, 29th October, 1958.

ADMISSION BY INVITATION ONLY — SEE BELOW

### BANQUET

The Annual Dinner of the Branch will be held on Friday, 31st October, 1958 at the Midland Hotel, Manchester.

## EXHIBITION

### "THE DIVERSITY OF STRUCTURAL ENGINEERING"

to be held in the

EXHIBITION HALL, COLLEGE OF SCIENCE AND TECHNOLOGY MANCHESTER

10.30 to 7.30 Monday, 27 October to Friday, 31 October 10.30 to noon Saturday, 1 November

FILMS TO BE SHOWN DURING THE MORNING, AFTERNOON AND EVENING

*Symposium* : applications for registration (5s. including buffet lunch and transport between sessions) to M. D. Woods, A.M.I.Struct.E., 8, Dennison Road, Cheadle Hulme, Cheshire.

*Inaugural Meeting* : tickets (5s.) from H. J. Dowling, A.M.I.Struct.E., 82, Station Road, Hadfield, nr. Manchester.

*Banquet* : tickets (2 gns. each) from J. L. Robinson, A.M.I.Struct.E., Department of Structural Engineering, College of Science and Technology, Manchester 1.

*Civic Reception* : applications (Branch members only) for limited invitation cards to J. L. Robinson at the above address.

### NORTHERN COUNTIES BRANCH

A meeting will be held in the Neville Hall, Newcastle, on Wednesday, 1st October, 1958, when the Chairman-Elect, Mr. D. W. Portus, M.I.Struct.E., will give an address.

Mr. Portus will be installed as Chairman of the Branch on Tuesday, 21st October, 1958, at the Cleveland Scientific and Technical Institution, Middlesbrough, and will give his Chairman's Address.

The above meetings will commence at 6.30 p.m., preceded by buffet tea at 6 p.m.

*Hon. Secretary* : H. W. Dowe, A.M.I.Struct.E., 2, The Crescent, Saltburn-by-the-Sea, Yorks.

### NORTHERN IRELAND BRANCH

*Hon. Secretary* : L. Clements, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E., 3, Kingswood Park, Cherryvalley, Belfast.

### SCOTTISH BRANCH

The opening meeting of the Session will be held at the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, on Monday, 13th October, at 7 p.m., when the Chairman's Address will be given by Mr. W. Heigh, M.I.Struct.E.

The Annual Dinner and Dance will be held at the Grosvenor Restaurant, Gordon Street, Glasgow, on Tuesday, 14th October, at 6.30 p.m.

*Hon. Secretary* : W. G. Cantlay, B.Sc., A.M.I.Struct.E., A.M.I.C.E., 3, Blairbeth Terrace, Burnside, Glasgow.

### SOUTH WESTERN COUNTIES BRANCH

*Hon. Secretary* : C. J. Woodrow, J.P., "Elstow," Hartley Park Villas, Mannamead, Plymouth, Devon.

### WALES AND MONMOUTHSHIRE BRANCH

*Hon. Secretary* : K. J. Stewart, A.M.I.Struct.E., A.M.I.C.E., 15, Glanmor Road, Swansea.

### WESTERN COUNTIES BRANCH

*Hon. Secretary* : A. C. Hughes, M.Eng., A.M.I.Struct.E. A.M.I.C.E., 21, Great Brockeridge, Bristol, 9.

### YORKSHIRE BRANCH

The opening meeting will be held at the Metropole Hotel, King Street, Leeds, on Wednesday, 15th October when the Chairman's Address will be given by Professor R. H. Evans, C.B.E., D.Sc., Ph.D., M.I.Struct.E., M.I.C.E., M.I.Mech.E. The Address will be repeated at a meeting to be held at the Royal Victoria Hotel, Sheffield, on Thursday, 16th October.

The above meetings will commence at 6.30 p.m., preceded by buffet tea at 6.15 p.m.

*Hon. Secretary* : W. B. Stock, A.M.I.Struct.E., 34, Hobart Road, Dewsbury, Yorks.

### UNION OF SOUTH AFRICA BRANCH

*Hon. Secretary* : A. E. Tait, B.Sc., A.M.I.Struct.E., A.M.I.C.E., P.O. Box 3306, Johannesburg, South Africa.

During weekdays Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. Phone 34-1111, Ext. 257.

*Natal Section Hon. Secretary* : J. C. Panton, A.M.I.Struct.E., A.M.I.C.E., c/o Dorman Long (Africa) Ltd., P.O. Box 932, Durban.

*Cape Section Hon. Secretary* : R. F. Norris, A.M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

### EAST AFRICAN SECTION

*Chairman* : R. A. Sutcliffe, M.I.Struct.E.

*Hon. Secretary* : K. C. Davey, A.M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

### SINGAPORE AND FEDERATION OF MALAYA SECTION

The following Honorary Officers and Committee members have been elected for the Session 1958-59 :—

*Chairman* : Mr. T. Karmakar.

*Hon. Secretary* : Mr. W. N. Cursitor, B.Sc., A.M.I.Struct.E., c/o Redpath Brown & Co. Ltd., P.O. Box 648, Singapore.

*Acting Hon. Secretary* : Mr. Chin Fung Kee, Department of Engineering, University of Malaya, Singapore, 10.

*Hon. Treasurer* : Mr. G. C. Chou.

*Committee* : Messrs. Lee Tuh-Fuh, Albert P.C. Wong, K. N. Lekshmanan, Tan Chin Thye, Tsang Foh Kwei, Neoh Poh Choon, H. S. Bull, Wee Soo Bee.

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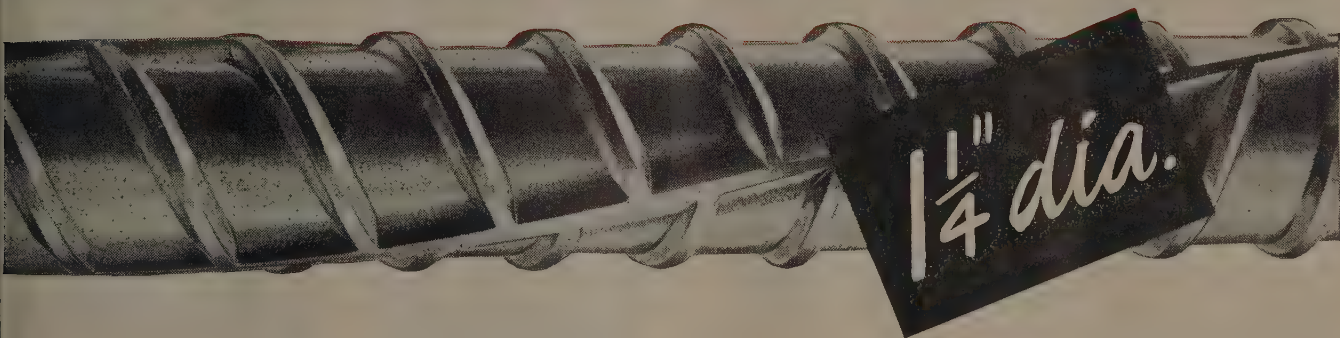


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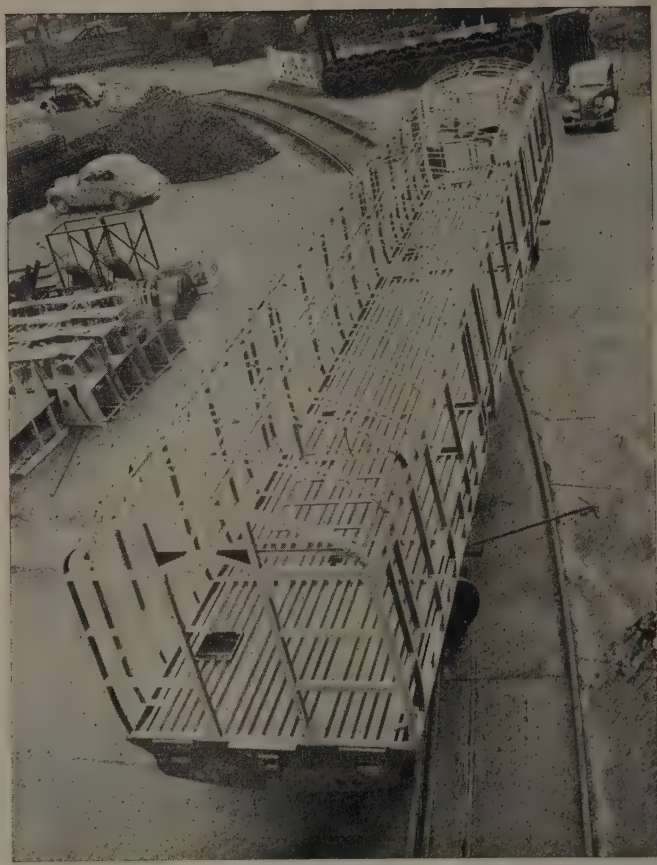

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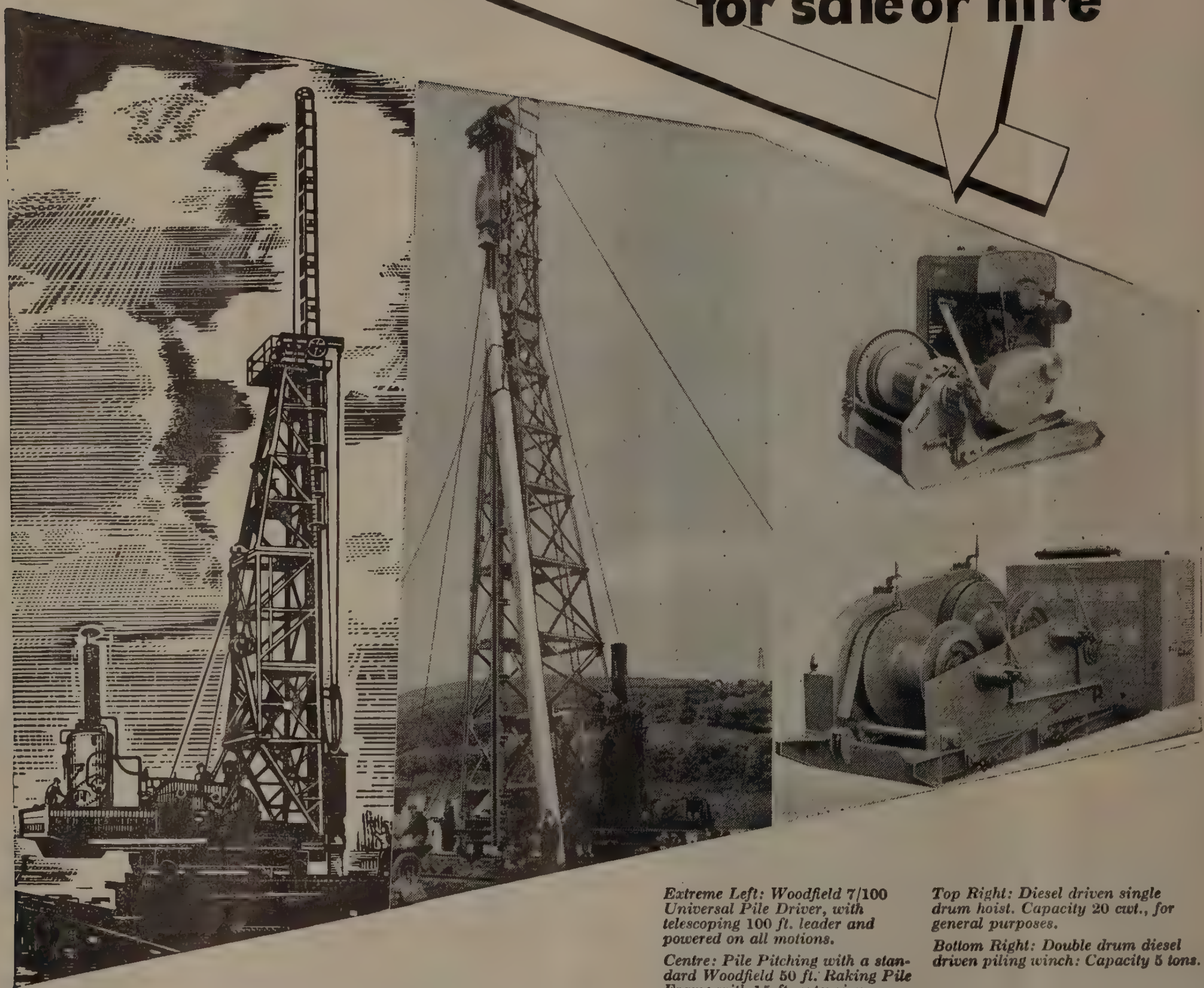

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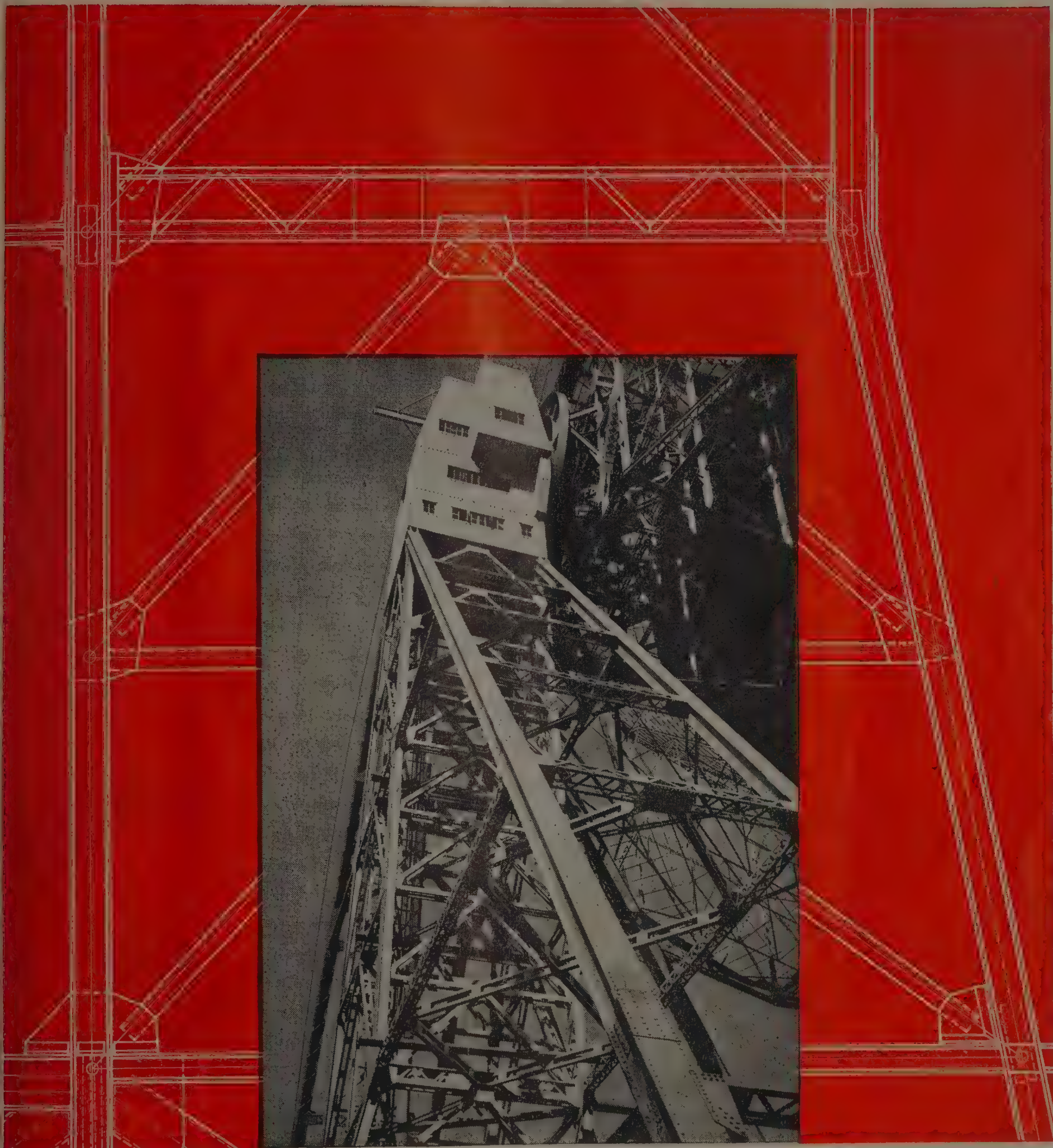

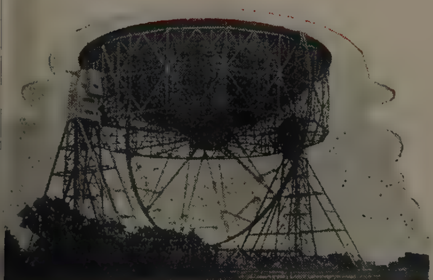

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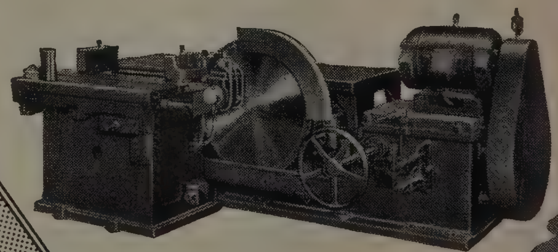
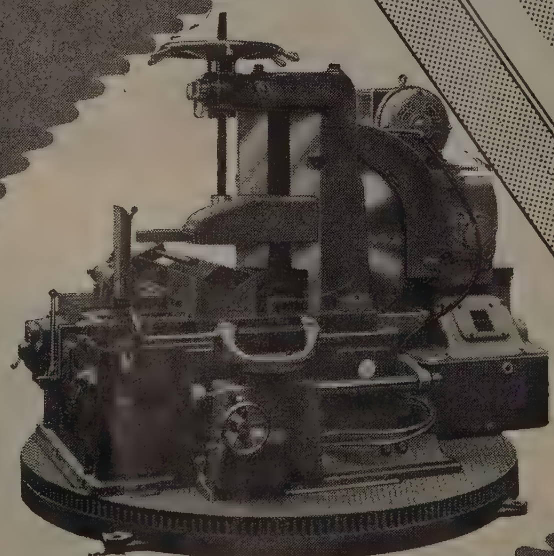


Horizontal Table Type.



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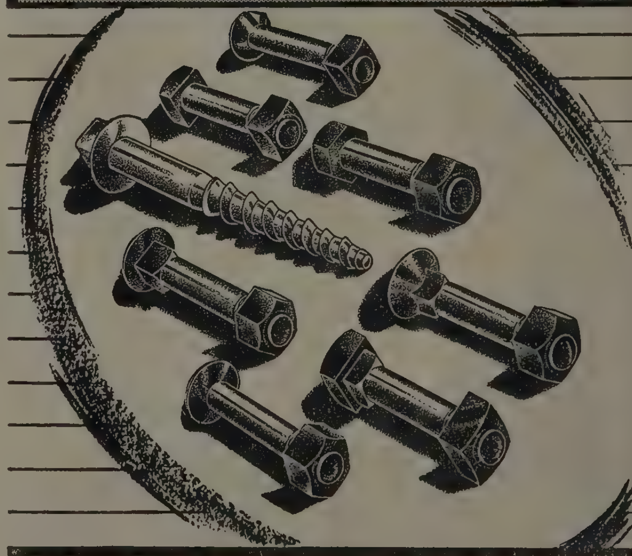

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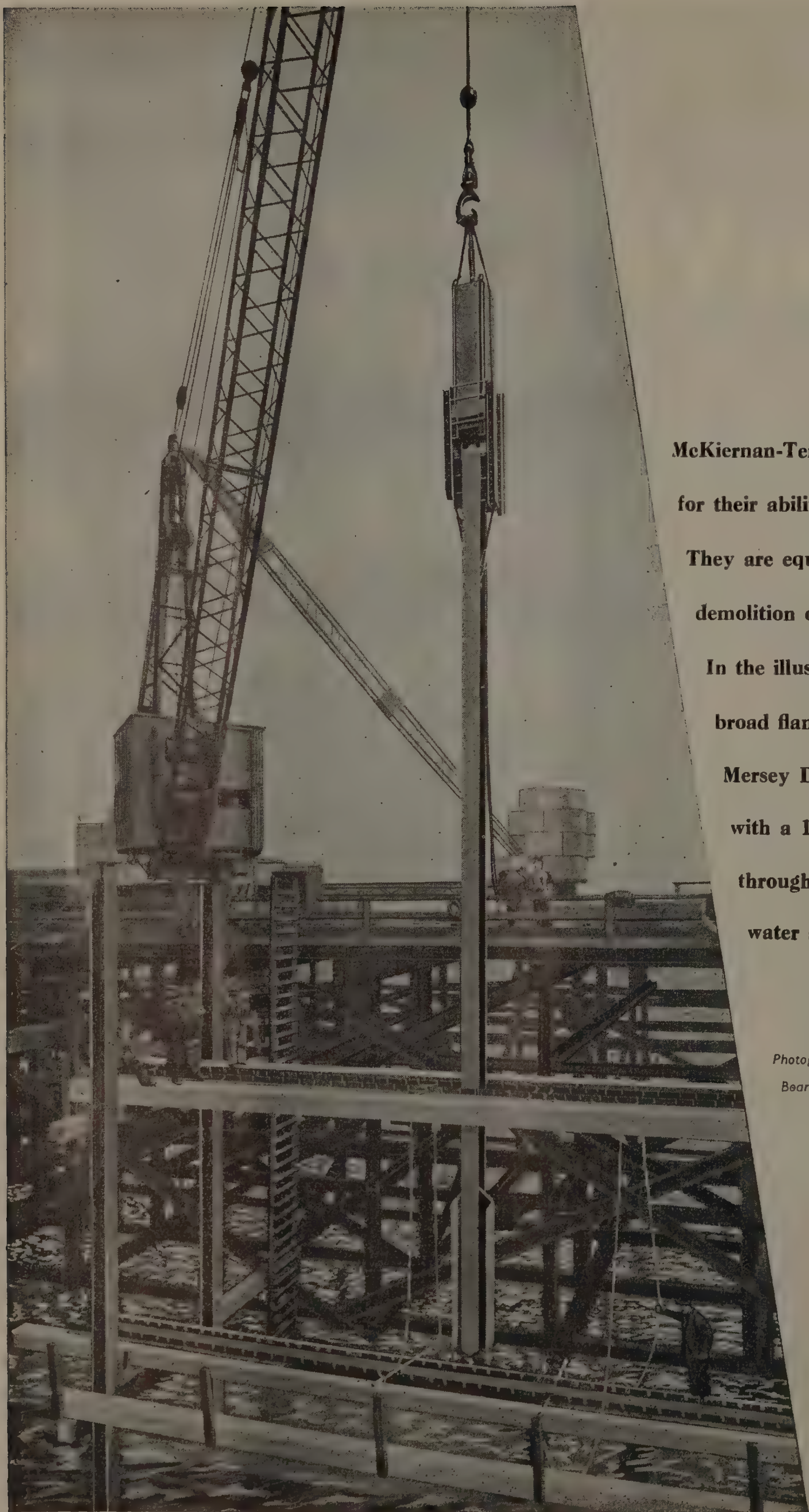

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## OFFICIAL APPOINTMENTS

**BRITISH** Railways—Western Region. Applications are invited for two posts of Senior Engineering Assistants in the Civil Engineering Department at Paddington Station, experienced in the design of girder bridges. Workshop experience desirable. Salary range £1,015-£1,065. Travel concessions, holidays with pay, prospects of promotion and permanency, membership of Superannuation Fund.—Apply stating age, previous experience, etc. to Chief Civil Engineer, B.R. Western Region, Paddington Station, W.2.

**BRITISH** Railways—Western Region. Applications are invited for the post of Engineering Assistant in the Civil Engineering Department at Paddington Station, experienced in design of prestressed and reinforced concrete bridges and other structures. Salary range £916-£956. Travel concessions, holidays with pay, prospects of promotion and permanency, membership of Superannuation Fund.—Apply stating age, previous experience, etc. to Chief Civil Engineer, B.R. Western Region, Paddington Station, W.2.

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**CITY** and County of Newcastle upon Tyne City Architect's Department. Applications are invited for the post of Senior Structural Engineer in the City Architect's Department in Scale 'B', at a salary of £1,210 per annum rising by annual increments to a maximum of £1,440 per annum. Applicants should preferably be Associate-Members of the Institution of Structural Engineers or the Institution of Civil Engineers. The officer appointed will be responsible for the preparation of calculations and detailed structural schemes for steel and reinforced concrete framed buildings, and experience in dealing with problems of special foundations will be an advantage. The above appointment will be subject to the provisions of the Local Government Superannuation Acts, 1937-53, and to three month's notice on either side. The successful candidate will be required to pass a medical examination.—Further particulars and Forms of Application may be obtained from George Kenyon, A.R.I.B.A., A.M.T.P.I., City Architect, 18, Cloth Market, Newcastle upon Tyne 1. John Atkinson, Town Clerk, Town Hall, Newcastle-upon Tyne 1.

**D.S.I.R.** requires two Scientific Officers (Ref. E.274/8A) and three Assistant Experimental Officers (Ref. E275/8A) at the Road Research Laboratory, Harmondsworth, Middx., in the Colonial Section which advises Overseas Governments on road and traffic engineering and undertakes research into road problems of overseas territories. Opportunities of visits overseas. Facilities available for obtaining wide knowledge and experience of road engineering at home and abroad. S.O.1. Civil Engineer for construction and maintenance of low cost roads, Soil Stabilisation, and bituminous surfacing materials. 2. Physicist for traffic engineering, with special reference to overseas problems, *A.E.O.* Two men will join the group concerned with soil problems in road construction overseas, and will be initially members of a small team investigating the mechanism of moisture movements in soils under roads. One man will be concerned with work on traffic and safety. Quals. S.O. 1st or 2nd Hon. degree in appropriate subject *A.E.O.* G.C.E. (Advanced) in Physics and one other science or maths subject. Over age 22 pass degree, H.N.C. in Physics, or equivalent generally expected. *Salary S.O.* in range £615-£1,080 (men). Normal prospects of promotion to *P.S.O.* (range up to £2,000) in mid thirties. A.E.O. £377 10s. 0d. (age 18)-£825 (men). Opportunities for promotion to E.O. (up to £1,160) and S.E.O. (up to £1,555). Women slightly less. Five day week, good leave allowance.—Forms from M.L.N.S., Technical and Scientific Register (K), 26, King Street, London S.W.1, quoting appropriate reference. Closing date 15th September, 1958.

OFFICIAL APPOINTMENTS—continued

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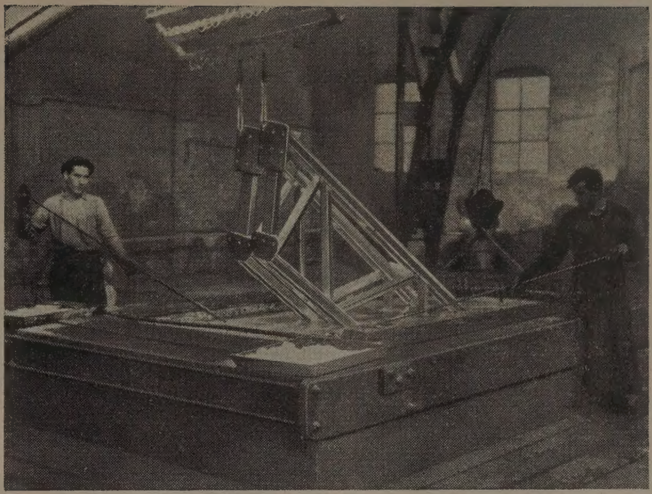

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